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Chapter Forty-nine

ROADSIDE SAFETY

49-1.0 GENERAL

49-1.01 Clear Zone Concept

The ideal roadway would be free from obstructions or hazardous conditions within the entire right-of-way. This is usually not practical due to economic, environmental or drainage needs. The clear zone concept was developed as a guide to determine how much obstruction-free recovery area should be provided for run-off-the-road vehicles. The clear zone width estimates provided here, as derived from the AASHTO *Roadside Design Guide*, provide adequate space for approximately 80% of the drivers who run off the road to gain control of their vehicles. It is important to note that the clear zone widths are only approximate values. It is the designer's responsibility to use good judgement, based on accident data when available, to determine if hazardous roadside features, including those outside the clear zone, warrant some type of treatment.

49-1.02 Special Situations Requiring Greater Width

The basic clear zone values assume a tangent roadway section and level or near level roadside slopes. Steeper down slopes require greater clear zone widths because a vehicle requires more distance to stop or turn on a down slope. Therefore, the horizontal width of the clear zone on a down slope must be extended to be equivalent to a level clear zone. Likewise, sharp horizontal curves, the location of non-traversable drainage ditches and similar situations affect the area alongside the roadway defined as a recovery area for the errant vehicle. It is equally apparent that a slower speed vehicle encroaching upon the roadside would not travel as far from the edge of the travel lane as one operating at a higher speed.

49-1.03 Applicability

The clear zone requirements provided here apply only to projects on new location, reconstruction projects and 3R and partial 4R projects on freeways. The roadside safety requirements for 3R non-freeway projects are presented in Section 55-5.0.

Wherever reference is made to speed, it is intended that the design speed be used. Design speeds for new construction/reconstruction projects are provided in Chapter Fifty-three. Design speeds for 3R and partial 4R projects on freeways are provided in Chapter Fifty-four.

Wherever reference is made to ADT, it is intended to be the design year traffic volumes which are typically assumed to 20 years in the future; see Section 40-2.02.

49-1.04 Right-of-Way

Right-of-way (R/W) is established to clear the construction limits. The construction limits are determined using a cross section that is traversable out to the R/W line or to the end of the clear zone, whichever is closer to the edge of the travel lane. Reducing R/W widths by designing steep embankment slopes that will require the installation of guardrail should be avoided unless necessary due to restricted conditions (e.g., environmental, dense development).

49-1.05 Cost-Effectiveness of Safety Improvements

Warrants for countermeasures should be in accordance with the appropriate sections in this Chapter. The cost-effectiveness of various countermeasures for hazardous roadside conditions should desirably be considered. Therefore, the designer is encouraged to use the ROADSIDE computer program described in Section 49-10.0 as a tool in selecting an alternative safety treatment which offers the greatest anticipated return of safety benefits for the funds expended. ROADSIDE can be used to evaluate many of the safety treatments outlined in this Chapter to determine if they are cost effective at a specific location. ROADSIDE should not be used to determine whether or not countermeasures are warranted at a particular location. Engineering judgment must be used in applying the results from ROADSIDE.

49-1.06 Adherence to Design Criteria

The designer should make every reasonable effort to meet the design criteria presented within this Chapter (e.g., clear zones, barrier length of need). However, if this is not practical, then a Level Two design exception is required. The designer will document in the project file that the design criteria have not been met and provide a brief rationale for not meeting the criteria. It will not be necessary to prepare in-depth documentation to justify the decision. ROADSIDE can be used as part of the required documentation justification. Section 40-8.0 further describes the Department's design exception procedures.

All new installations of guardrail, impact attenuators and other safety hardware should meet the placement and installation criteria presented within this Chapter and the *INDOT Standard Drawings*.

Environmental mitigation measures should not supercede roadside safety criteria. However, environmental mitigation features may be considered and incorporated into the project consistent with the criteria presented within this chapter.

49-2.0 ROADSIDE CLEAR ZONES

<u>49-2.01 Clear Zone Distances</u>

Figure 49-2A, Clear Zone Distances (m) (New Construction/Reconstruction), presents the clear zone distances for design. These distances are estimates of the traversable area required adjacent to the edge of the travel lane and are based on a set of curves from the 1988 AASHTO *Roadside Design Guide*. These curves are for tangent sections and various side slopes. They were developed assuming essentially an infinite length of side slope and 3.6-m shoulders. Intervening ditches or multiple slopes require special consideration.

Referring to Figure 49-2A for a given side slope and design year ADT, the appropriate clear zone width for a given design speed can be determined. For example, for a highway with a design speed of 100 km/h, an ADT of 5000 vehicles and a 4:1 fill slope, the suggested clear zone is 12.5 m. For a 4:1 cut slope, the required clear zone is 6.0 m.

A basic understanding of the clear zone concept is critical to its proper application. The numbers obtained from Figure 49-2A imply a degree of accuracy that does not exist. The numbers are based on limited empirical data which was then extrapolated to provide data for a wide range of conditions. Thus, the numbers obtained are neither absolute nor precise. They do, however, provide a good frame of reference for making decisions on providing a safe roadside area.

In applying the clear zone values, the designer should consider the following:

1. <u>Context</u>. The clear zone values, in Figure 49-2A are not absolute numbers. It is desirable to eliminate all hazards within the R/W; however, in most cases this is not practical because of economic or environmental constraints. In some cases, it is reasonable to leave a fixed object within the clear zone; in other cases, an object beyond the clear zone distance may warrant removal or shielding. The use of an appropriate clear zone distance is a compromise between maximum safety and minimum construction costs. The designer should use good engineering judgement when determining if a roadside hazard should be removed or shielded if it is outside the clear zone but within the right of way.

2. <u>Adjustments</u>. The clear zones in Figure 49-2A can be used for roadways which have shoulders less than 3.6 m in width without applying any adjustment factors. The clear zone is still measured from the edge of the travel lane, and slope averaging starts at the edge of

shoulder.

3. Right-of-Way. If the clear zone falls outside the R/W, use the distance from the edge of the travel lane to the R/W line as the clear zone width.

4. Guardrail. Where guardrail is required, these clear zones are used to determine the length of guardrail need.

5. Boundaries. The designer should not use the clear zone distances as boundaries for introducing roadside hazards such as bridge piers, non-breakaway sign supports, utility poles or landscape features. These should be placed as far from the roadway as practical.

6. Design Year ADT. For clear zones, the "Design Year ADT" will be the total ADT on two-way roadways and the directional ADT on one-way roadways. Examples of one-way roadways include ramps and the directional roadway of a divided highway.

49-2.02 Clear Zone Adjustments

The clear zone should not vary with small variations in highway features. It should be constant on a length of road with a fairly consistent roadside. For highways on new location, the clear zone should be constant for as much of the length of project as practical.

49-2.02(01) Horizontal Curve Correction

Horizontal curves increase the angle of exit from the roadway and thus increase the width of clear zone required. Figure 49-2B, Clear Zone Adjustment Factors for Horizontal Curves (K_{cz}), provides horizontal curve correction factors that should be applied to the tangent clear zone widths to adjust them for roadway curvature. Figure 49-2C illustrates the application of the adjusted clear zones on curves. It should be noted that curves with radii greater than 875 m as measured along the roadway centerline, will not require a curvature adjustment. The horizontal curve correction is required on all new construction and reconstruction projects and on all 3R and partial 4R freeway projects. If the correction cannot be practically applied, then a Level Two design exception will be required for these projects.

The transition between different width clear zones along highways with tangents and curve radii greater than 875 m should be applied as shown in Figure 49-2D, Clear Zone Transition for Tangent Sections and Curves with Radius > 875 m. The transition lengths between the beginning and the end of the narrow and wider clear zones may vary.

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Example 49-2.1

Given: Rural Collector
Design Speed = 90 km/h
2000 Design Year ADT
Horizontal curve with a radius of 600 m
3:1 cut slope

Problem: Find the adjusted clear zone.

Solution: From Figure 49-2A, the clear zone on the tangent (CZ_t) = 4.5 m.

From Figure 49-2B, the curve correction factor (K_{cz}) = 1.2
Clear zone for the curve (CZ_c) = 4.5 m *x* 1.2 = 5.4 m (assume 5.5 m)

The transition length (L) = .6 *x* 90 = 54 m.

* * * * * * * * * *

49-2.02(02) Slope Averaging

Variable fill slopes have often been used along roadways to provide a relatively flat recovery area immediately adjacent to the roadway followed by a steeper side slope. Clear zone distances for embankments with variable side slopes ranging from essentially flat to 4:1 may be averaged, using a "weighted average" within the clear zone, to produce a composite clear zone distance. The slope averaging should begin at the outside edge of the adjacent travel lane for opposing traffic; see Figure 49-2E, Slope Averaging (Example).

Slope averaging only applies to slopes in the same direction. Slopes which change from a "down" slope to an "up" slope, as for a ditch section, cannot be averaged and should be treated as discussed in Section 49-2.03(01).

<u>49-2.03 Clear Zone Applications</u>

49-2.03(01) Roadways with Shoulders/Mountable Curbs (V $\geq$ 60 km/h)

This Section applies to all projects on freeways, including 3R and partial 4R projects and to all new construction and 4R projects on rural and urban arterials, and rural and urban collectors with design speeds of 60 km/h or more. Section 49-2.03(02) provides the clear zone applications for rural and urban collectors with design speeds less than 60 km/h, rural local roads and urban local streets.

Section 49-2.03(03) provides the clear zone applications for urban facilities with barrier curbs.

The designer should consider the following clear zone applications for the above-listed facilities with shoulders or mountable curbs:

1. Criteria. The clear zone distances presented in Figure 49-2A with the appropriate adjustments from Section 49-2.02 should be used.

2. Fill Slopes (Reconstruction Projects). To calculate the recommended clear zone distance on reconstruction projects with fill slopes, the designer should consider the following:

   a. Figures 49-2A and 49-2B, with the applicable design speed, ADT and foreslope, are used to determine the appropriate clear zone distance. If the clear zone falls outside the right-of-way, use the right-of-way line as the clear zone distance.

   b. For variable fill slopes 4:1 or flatter, use a weighted average as discussed in Section 49-2.02(02) to determine the slope, then proceed as discussed in Item #2.a above.

   c. Fill slopes steeper than 4:1 are considered non-recoverable and should not be included in slope averaging. When a vehicle encroaches onto a non-recoverable slope, it can be assumed that the vehicle will continue to travel to the bottom of the slope. Therefore, if the clear zone distance extends onto the non-recoverable slope, a clear runout area should be provided at the bottom of the slope. This clear runout area should be equal in width to the portion of the clear zone distance which extends onto the non-recoverable slope or 3.6 m, whichever is greater. See Figure 49-2F, Clear Zone Application for Fill Slopes (Non-Recoverable Slope), for an illustration of this procedure.

3. Fill Slopes (New Facilities). Desirably for new facilities, a 6:1 fill slope as shown in Figure 49- 2G, Clear Zone Application for Side Slopes (New Facilities), should be used adjacent to the roadway. At a minimum, the criteria as described for reconstruction projects in Item 2 above may be used.

4. Cut Slopes (Reconstruction Projects). To calculate the recommended clear zone on reconstruction projects with cut slopes, the designer should consider the following:

   a. If a ditch is traversable, use Figure 49-2A with the applicable design speed and ADT to check the clear zone distance required for the foreslope and the backslope. Generally, the foreslope clear zone will control. However, if the toe of the backslope is within 3.0 m of the shoulder edge, the clear zone for the backslope should be used. See Section 49-3.0 to determine if the ditch is traversable.

b. If the ditch is not traversable, the ditch should be reconstructed to a section which is traversable. The clear zone is then calculated as in Item 4.a above.

c. Cut slopes of 2:1 are not desirable. However, if they will be retained or constructed within the clear zone, then the ditches in front of them should be made traversable. Figure 49-2H, Clear Zone Application for Cut Slopes (2:1 Backslopes), illustrates the desirable cross section if a 2:1 backslope will be retained. If it is not practical to construct a traversable ditch, the designer should consider the accident experience at the site and use engineering judgment to determine if guardrail is warranted.

5. Cut Slopes (New Facilities). Desirably for new facilities, a ditch section as shown in Figure 49-2G should be used. At a minimum, the criteria as described in Item 4 above for reconstruction projects may be used. However, 2:1 backslopes on new facilities should not be used.

6. Auxiliary Lanes. Adjacent to acceleration and deceleration lanes, existing slopes should be measured by averaging the slopes from the edge of the theoretical 3.6-m shoulder. The clear zone is measured from the edge of the through travel lane, and it is based on the mainline ADT and design speed. The clear zone should also be checked for the auxiliary lane using the auxiliary lane ADT and mainline design speed. In the latter case, the clear zone is measured from the outside edge of the auxiliary lane. Example 49-2.2 illustrates an example calculation of the clear zone from the edge of the through lane using slope averaging. Figure 49-2 I, Clear Zone Applications (Auxiliary Lanes and Ramps), illustrates the typical clear zone application for an auxiliary lane next to the mainline.

7. Ramps. If the obstacle is adjacent to a ramp, the clear zone should be determined the same as for the mainline, using the ADT and design speed of the ramp and the slope from the ramp shoulder. Figure 49-2 I illustrates the typical clear zone application for a ramp/mainline configuration.

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Example 49-2.2

Given: Rural freeway with an exit ramp
7000 Design Year ADT
Design speed = 110 km/h
A 3.6-m wide deceleration lane with a 2.4-m right shoulder
A 4:1 slope adjacent to deceleration lane shoulder

Problem: Determine the clear zone adjacent to the deceleration lane.

Solution: Start slope averaging from edge of theoretical shoulder; see Figure 49-2J, Clear

Zone/Slope Average (Example 49-2.2).

First Trial: Assume clear zone for the mainline ends 3.0 m beyond the deceleration lane shoulder.

Therefore, assumed clear zone = 3.6 + 2.4 + 3.0 = 9.0 m

$$\text{Slope} = \frac{(2.4)(-0.04) + (3.0)(-0.25)}{5.4} = \frac{(-0.10) + (-0.75)}{5.4} = 0.16 \text{ or } 6:1 \text{ slope}$$

From Figure 49-2A, the clear zone = 10.5 m

10.5 m > 9.0 m; therefore, a second trial is necessary with a larger assumed clear zone.

Second Trial: Assume clear zone ends 6.0 m from existing shoulder.

Therefore, assumed clear zone = 3.6 + 2.4 + 6.0 = 12.0 m

$$\text{Slope} = \frac{(2.4)(-0.04) + (6.0)(-0.25)}{8.4} = \frac{(-0.10) + (-1.50)}{8.4} = 0.19 \text{ or approximately } 5:1$$

From Figure 49-2A, the clear zone = 11.5 m.

12.0 m is close enough to 11.5 m; therefore, 11.5 m is the required clear zone from the edge of the through travel lane.

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49-2.03(02) Roadways with Shoulders/Mountable Curbs (V < 60 km/h)

This Section applies to all new construction and reconstruction projects on rural and urban collectors with design speeds less than 60 km/h, and to all local roads and streets. Desirably, the clear zone should be determined from Figure 49-2A, Clear Zone Distances (m) (New Construction/Reconstruction), with the applicable adjustments. The minimum clear zone is 3.0 m for tangent sections and should be adjusted as discussed in Section 49-2.02 for horizontal curves. Where the clear zone extends onto a 3:1 fill slope, a clear recovery area as shown in Figure 49-2F, Clear Zone Application for Fill Slopes (Non-Recoverable Slope), should be provided.

49-2.03(03) Roadways with Barrier Curbs

For urban arterials, collectors and local streets with barrier curbs at either the edge of the travel lane or the edge of shoulder, the minimum clear zone is 3.0 m from the edge of the travel lane or to the right-of-way line, whichever is less.

49-2.03(04) Appurtenance-Free Area

Roadways for all functional classifications should have a 0.5-m appurtenance-free area from the face of curb or from the edge of the travel lane if there is no curb. However, for traffic signal supports, the appurtenance-free area should be 0.8 m. The appurtenance-free area is defined as a space in which nothing, including breakaway safety appurtenances, should protrude above the paved or earth surface (see Figure 59-2K, Appurtenance Free Zone). The objective is to provide a clear area adjacent to the roadway in which nothing will interfere with extended side-mirrors on trucks, with the opening of vehicular doors, etc.

49-2.03(05) On-Street Parking

The following clear zone requirements will apply to facilities with on-street parking.

1. <u>Continuous 24-Hour Parking</u>. No clear zone is required on facilities where there is continuous 24-hour parking, except that the appurtenance-free area of 0.5 m should be provided from the face of the curb or edge of the parking lane if there is no curb.

2. <u>Parking Lane Used as a Travel Lane</u>. The clear zone should be determined assuming the edge of the parking lane as the right edge of the farthest right travel lane.

49-3.0 TREATMENT OF OBSTRUCTIONS

<u>49-3.01 Roadside Hazards</u>

49-3.01(01) Range of Treatments

Obstructions and non-traversable hazards within the clear zone should be, in order of preference, as follows:

1. removed or redesigned so that they can be safely traversed,
2. relocated outside of the clear zone to a point where they are less likely to be hit,
3. made breakaway to reduce impact severity,
4. shielded with a traffic barrier or impact attenuator, or
5. delineated if the above treatments are not practical.

49-3.01(02) Example Hazards

The method for treating the obstruction should be based on an analysis of several factors such as initial cost, maintenance cost, and the greatest safety return. The following is a list of some of the obstructions and hazards which should be considered for treatment.

1. non-breakaway sign supports;

2. non-breakaway luminaire supports (Note: Sign and luminaire supports in the clear zone should not be placed on breakaway supports if there is a sidewalk and there is a potential for these supports falling on pedestrians or bicyclists);

3. bridge piers;

4. bridge rail ends (Note: Bridge rail ends must have appropriate approach guardrail even if end is outside clear zone);

5. ends of all culverts which are transverse to the mainline road and do not have an acceptable end treatment in accordance with Section 49-3.03;

6. all concrete headwalls on culverts;

7. all trees;

8. retaining wall ends;

9. mailbox supports (Note: All mailbox supports should be placed in accordance with the INDOT *Standard Drawings*, INDOT *Standard Specifications* and Section 51-11.0);

10. wood poles or posts with a cross sectional area greater than 0.015 m^2;

11. utility poles (Note: Utility poles should be installed as close as practical to the right-of-way line);

12. standard steel pipe with an inside diameter greater than 50 mm;

13. large boulders;

14. rough rock cuts;

15. bridge cone slopes that are 2:1 or steeper and can be hit head-on;

16. severely rutted or eroded slopes;

17. transverse embankment slopes for driveways, public road approaches, ditch checks and median crossovers that are steeper than those shown in Figure 49-3A, Transverse Slopes, for various design speeds and ADT levels;

18. ditch cross-sections that do not conform to the criteria presented in Section 49-3.02;

19. streams or bodies of water where the permanent water depth is 0.6 m or greater; and/or

20. slopes steeper than 1:1 at the edge of shoulder and a height greater than 0.6 m.

49-3.02 Roadside Ditches

49-3.02(01) General Guidelines

Traversable ditch cross sections are defined in Figure 49-3B, Preferred Cross Sections for Ditches (Narrow-Width Ditches), Figure 49-3C, Preferred Cross Sections for Ditches (Medium-Width Ditches), and Figure 49-3D, Preferred Cross Sections for Ditches (Wide-Width Ditches). Two curves are shown on each figure. The area below the lower curve represents ditch cross sections which can be traversed by a vehicle containing unrestrained occupants and, thus, are considered to be desirable. Ditch cross sections which are between the upper curve and the lower curve are considered to be acceptable. However, vehicle occupants must be restrained in order to safely traverse the ditch. Minor encroachment into the area above the upper curve may be necessary due to right-of-way restrictions or to avoid nominal changes to existing ditches. In addition, the following should be considered.

1. Slopes of 3:1 should be used only where site conditions do not permit the use of flatter slopes.

2. To permit traversability of a 3:1 slope, embankment surfaces should be uniform. Vehicular rollover can be expected if the embankment is soft or rutted.

3. Foreslopes steeper than 4:1 are not desirable because their use severely limits the range of backslopes producing a safe ditch configuration.

49-3.02(02) Application

If the ditch falls outside the clear zone, the designer is not required to check the ditch for traversability. For ditches within the clear zone, the following describes the appropriate application of Figure 49-3B, 49-3C, or 49-3D.

1. In Fills (Reconstruction Projects). Existing ditch slope combinations which fall within the desirable or acceptable range may be retained. Areas with ditch slope combinations which fall within the undesirable range should be evaluated for cost and accident history before deciding to make an improvement. If an improvement is warranted, the slope combination should preferably fall within the desirable range and at least within the acceptable range.

2. In Fills (New Facilities). The designer should select a foreslope, backslope and ditch width that will fall within the desirable range in Figure 49-3B, 49-3C, or 49-3D.

3. In Cuts (Reconstruction Projects). If the ditch is such that, to flatten the slopes or moving the ditch out farther means acquiring more right-of-way, then this should be done only if considered to be cost effective. Other means of making the ditch traversable which can be evaluated are as follows:

    a. use of a pipe in the ditch,

    b. raise the grade of the ditch, or

    c. place 100-mm rip rap in the ditch to change the ditch contour (rip rap must be placed so that projection of any one piece of rip rap does not exceed 50 mm above surrounding area).

4. In Cuts (New Facilities). The desirable ditch section is shown in Figure 49-2G. For minimum ditch sections, the designer should provide a section which falls within the desirable range in Figure 49-3B, 49-3C, or 49-3D.

49-3.03 Drainage Structures

49-3.03(01) Cross Drainage Structures

The following provides the Department's criteria for drainage structures which are perpendicular or skewed to the roadway centerline and have culvert ends that are within the clear zone.

1. 300-mm Culverts. These pipe structures and equivalent pipe arch culverts should use a standard metal culvert end section as shown in the INDOT *Standard Drawings*.

2. 375-mm to 1500-mm Culverts (10-Deg Skew or Less). These pipe structures and equivalent pipe arch culverts should be installed with a safety metal culvert end section (or

an optional grated box end section) as shown in the INDOT *Standard Drawings*. For areas with side slopes of 3:1 or steeper, culverts from 375 mm to 750 mm diameter may use a standard metal culvert end section. For areas with side slopes of 3:1 or steeper, culverts from 900 mm to 1500 mm diameter may use a safety culvert metal end section (or an optional grated box end section). Type I grated box end sections should be used at high accident locations where it is anticipated that vehicles will most likely traverse them based on previous accident experience. This applies to all culverts except where the culvert end is behind guardrail having adequate length to shield the end from errant vehicles.

3. <u>375-mm to 1500-mm Culverts (Greater Than 10-Deg Skew)</u>. These pipe structures and equivalent arch culverts which are skewed more than 10 deg should have a grated box end section (GBES) installed perpendicular to the roadway centerline as shown in the INDOT *Standard Drawings*. This applies to all culverts except where the culvert end is behind guardrail having adequate length to shield the end from errant vehicles. Large skews may require the use of a GBES that is intended for a larger pipe in order to provide an adequate opening in the GBES for the skewed pipe.

   In some cases, it may be necessary to maintain the direction of flow in a straight line at the inlets and the outlets in order to perpetuate the channel flow. In these cases, the GBES must be installed parallel to the pipe centerline and the roadway embankment must be warped around the GBES to present a smooth slope profile.

4. <u>1675 mm and Larger Culverts</u>. If the culvert end is within the clear zone, guardrail should typically be provided; see Figure 49-3E, Large Culvert Ends within Clear Zones. If the culvert end falls outside the clear zone, guardrail should be placed to protect the errant motorist from the culvert end. If there is inadequate cover over the culvert to drive the guardrail posts, it will be necessary to use the detail for guardrail over low-fill culverts as shown in Section 49-5.03 and the INDOT *Standard Drawings*.

5. <u>Pipes in the Median</u>. Desirably, the adjoining ends of two transverse culverts in the median between divided travel lanes or between a main road and a frontage road should be connected if the ends are within the clear zone. At a minimum, pipes in the median should be treated the same as described above, except pipe structures 375 mm through 1500 mm should have a Type I grated box end section. Culverts with appropriate sloped grates should be installed in the parallel ditch as shown in Figure 49-3F, Culvert End Treatment (Median Sections).
6. <u>Box Culverts</u>. Simply extending a concrete box culvert beyond the clear zone is generally not acceptable. The most cost-effective treatment should be considered. This may include the following:

   a. removing sections of the box culvert and attaching metal circular or pipe arch adapters, a short section of metal culvert, and then an INDOT approved grated end section;

b. providing a specially designed grated end section; or

c. installing guardrail.

49-3.03(02) Parallel Drainage Structures

The following provides the Department's criteria for drainage structures which are parallel to roadway centerline and are within the clear zone.

1. 300-mm to 1500-mm Culverts in the Median. These pipe structures under cross-overs should be end fitted with Type II grated box end sections with a slope meeting the criteria shown in Figure 49-3A, Transverse Slopes.

2. 300-mm Culverts. These pipe structures and equivalent pipe arch culverts should use the standard metal culvert end section as shown in the *INDOT Standard Drawings*.

3. 375-mm to 1500-mm Culverts in Side Ditches. This includes both ends of a culvert on two-way roadways when both ends are within the clear zone for both the adjacent and opposing traffic or only to the end facing oncoming traffic on the outside of divided highways. It does not apply to the traffic downstream end of a culvert if it is outside the clear zone for opposing traffic. See Figure 49-3G, Culvert End Treatments (Parallel Structures).
   These pipe structures in the ditch line, parallel to the centerline, should be installed with a safety metal culvert end section. In areas requiring a 10:1 slope parallel to the roadway, the 10:1 slope may be warped to match the 6:1 slope of the safety metal culvert end section. Type II grated box end sections, with a slope as shown in Figure 49-3A, should be used at high accident locations where it is anticipated that vehicles will most likely traverse them based on previous accident experience. This applies to all culverts, except when the culvert end is behind guardrail having adequate length to shield the end from errant vehicles.

49-3.03(03) Inlets

The following presents the Department's criteria for the placement of drainage inlets within the clear zone.

1. General. Type-7 Inlets with vertical projections of 100 mm or greater should not be used in any new installations. Existing Type-7 Inlets should not be replaced unless their location is considered to be a safety hazard.

2. Reconstruction Projects. A Type E-7 Inlet in the median should not be replaced unless its location is considered to be a safety hazard. The Type E-7 Inlet should be replaced with an acceptable inlet type if the slopes adjacent to it must be regraded to eliminate a hazardous depression. If an existing Type E-7 casting is broken, it may be replaced.

3. New Facilities (or Reconstruction Projects). Only Type N-12 or P-12A Inlets will be allowed in the following situations.

    a. in medians in advance of the 20:1 slope grading for an attenuation device at a median pier or overhead sign structure support, or

    b. in a side ditch in advance of the 20:1 slope grading for a guardrail run that is buried in a backslope.

4. Interstates. On the Interstate system any Type N-12 or P-12 Inlet that does not have a 10:1 slope and is parallel to the centerline should be replaced with a new 10:1 slope Type N-12 or P-12A Inlet as shown in the INDOT *Standard Drawings*.

49-3.04 Curbs

49-3.04(01) General

In general, the use of curbs should be avoided. However, they are sometimes necessary to control drainage or to protect erodible soils. Section 45-1.05 and the INDOT *Standard Drawings* provide detailed information on the warrants and types of curbs used by the Department. When considering curbing relative to roadside safety, the designer should consider the following:

1. Design Speed. Facilities with a design speed greater than 70 km/h should be designed without curbs. However, if necessary, a 100-mm sloping curb may be used. Facilities with a design speed of 70 km/h or less may use either a sloping or vertical curb.

2. Roadside Barriers. The use of curbs with a roadside barrier is discouraged and, specifically, curbs higher than 100 mm should not be used with a barrier. Terrain conditions between the traveled way and a barrier can have significant effects on barrier performance. Curbs and sloped medians (including superelevated sections) are two prominent features which deserve special attention.

3. Redirection. It has been found that curbs offer no safety benefits on high-speed roadways on vehicular behavior following impact. Therefore, a curb should not be used for the purpose of redirecting errant vehicles.

49-3.04(02) Curbs on Ramps

Existing curbs on ramps should be removed and new stabilized shoulders should be constructed. Using 4.9 m as the pavement width for the ramp, the shoulders should be constructed such that a 1.2-m desirable, 0.8-m minimum stabilized shoulder is on the left side and a 2.4-m desirable, 2.3-m minimum stabilized shoulder is on the right side. If the existing pavement is more than 4.9 m in width, then that portion of the existing pavement over 4.9 m should be considered as part of the shoulders. For new facilities, see Section 48-5.0 and the INDOT *Standard Drawings*.

49-3.05 Bridge Piers and Spillslopes

49-3.05(01) New Construction Projects

The following presents the Department's criteria for bridge pier and spillslope clearance on new construction projects:

1. Divided Highways. On divided highways, the spillslope clearance should be equal to the clear zone of the approach roadway.

2. Vertical Clearance. After establishing the clear zone beneath an overhead structure, the critical vertical clearance must be determined. A critical vertical clearance of 4.3 m should be provided at the edge of the clear zone. The slope between the edge of shoulder and the edge of clear zone should be no steeper than 6:1. If the slope is steeper than 6:1, it should be flattened to 6:1 to provide a greater vertical clearance. See the following examples.

    a. Example 1. A county road crosses over a tangent freeway having a design speed of 110 km/h and a design year projected traffic count of 7500 ADT. From Figure 49-2A, Clear Zone Distances (m) (New Construction/Reconstruction), the minimum clear zone to the face of pier or toe of the 2:1 spillslope, assuming a 6:1 approach fill slope, is 10.5 m. See Illustration A in Figure 49-3H, Bridge Pier and Spillslope Clearance (New Construction). To maintain a minimum 4.3-m vertical clearance at the outer edge of the clear zone, the maximum permissible upward slope beyond the shoulder is 8:1 (cut section).

    b. Example 2. A county road crosses over a superelevated roadway having a design speed of 100 km/h, a design year projected traffic count of 1200 ADT and a horizontal curve with a 450-m radius. To hold the 4.3-m minimum vertical clearance at the outer edge of the clear zone, the maximum permissible slope beyond the shoulder line is 6:1 (upward) and 10:1 (upward) on the high side. See

Illustration B in Figure 49-3H.

Basic clear zone of approach roadway (low side - 6:1 fill)	= 7.5 m	(Figure 49-2A)
Basic clear zone of approach roadway (high side - 6:1 fill)	= 7.5 m	(Figure 49-2A)
Horizontal curve correction factor	= 1.4	(Figure 49-2B)
Horizontal clearance to pier or toe of 2:1 spillslope (low side)	= 7.5 m	
Horizontal clearance to pier or toe of 2:1 spillslope (high side)	= 7.5 m *x* 1.4 = 10.5 m	

Note that the curve correction factor is applied only to the outside (high side) of horizontal curves.

2. Shoulder Pier Clearance. The use of shoulder piers should be avoided wherever practical. However, if they are considered necessary, they should be placed as far from the edge of the traveled way as practical and shielded with guardrail as described in Section 49-3.05(02), if located within the clear zone.

3. Median Piers. All median piers will typically be shielded with guardrail and/or attenuators in accordance with the INDOT *Standard Drawings*.

49-3.05(02) Reconstruction Projects

On reconstruction projects where the piers or bridge cone spillslopes fall within the clear zone, the following procedures apply:

1. Slopewall Set Back 9.0 m from Edge of Travel Lane. Establish the elevation of the bottom of the slopewall. Below this elevation, the upstream bridge cone should be graded at a downward slope (equal to the slope below the concrete slopewall) to the intersection with the natural ground. This slope should be constructed between the edge of the bituminous paved apron and as close as practical to the right-of-way line. The built-up slope should be transitioned to the existing ground near the right-of-way line at a 4:1 or flatter slope. See Section 49-3.03 for culvert end treatment requirements.

   The area between the ends of the slopewall, and bounded by the edge of the paved shoulder and the base of slopewall, should be paved. At the downstream end of the paved apron, the new embankment should be graded at a 6:1 downward slope to meet the existing ground. Typical details are provided in Figure 49-3 I, Treatment at Existing Bridge Cones (Slopewall 9.0 m from Travel Lane).

2. <u>Slopewall Set Back Less Than 9.0 m from Edge of Travel Lane</u>. Spillslopes located less than 9.0 m from the travel lane should, in general, be graded in accordance with Figure 49-3J, Treatment at Existing Bridge Cones (Slopewall 3.0 m to 8.9 m from Travel Lane). The upstream bridge cone should be graded at a downward slope to intersect the natural ground. This slope should be constructed between the edge of slopewall and as close as practical to the right-of-way line; see Figure 49-3J. The built-up slope should be transitioned to the existing ground at a 4:1 or flatter slope. See Section 49-3.03 for culvert end treatment requirements. At the downstream end, the embankment should be graded at a 6:1 downward slope to meet the existing ground.

3. <u>Shoulder Piers</u>. Piers located within the clear zone should be protected with guardrail. A pier located within 4.9 m from the edge of the travel lane should be protected with a guardrail transition attached to the pier and the required length of guardrail. Piers located beyond 4.9 m but within the clear zone should be shielded with either a guardrail transition attached to the pier and the required length of guardrail, or a run of guardrail placed in front of the pier, as determined on the field check (see Sections 49-3.05(04) and 49-5.0). Where the run of guardrail is placed in front of the pier, the offset between the face of rail and the edge of the travel lane should be made as large as practical. The clearance between the back of the guardrail posts and the pier should be checked to conform with the guardrail deflection criteria. Figure 49-3K, Treatment at Existing Bridge Cones (With Shoulder Pier), provides typical details for shoulder pier protection.

   Where the offset distance between the face of pier and the edge of the travel lane is less than the minimum required usable shoulder width, a design exception will be required for the shoulder width even though the pier is protected with guardrail. A design exception will not be required when the face of pier is located beyond the minimum required usable shoulder width, and the guardrail transition projects into the shoulder area.

   The methods of treatment at existing piers and bridge cones described above and the details shown on Figures 49-3 I, 49-3J, and 49-3K provide satisfactory methods of treatment. Because actual field conditions are extremely variable, each location should be investigated carefully at the field check to determine if alternative solutions might be more acceptable.

4. <u>Median Piers</u>. All median piers should be shielded with guardrail and/or attenuators in accordance with Sections 49-3.05(05) and 49-6.0.

49-3.05(03) Longitudinal Side Slope Transitions

Section 45-3.0 presents the Department's criteria for fill and cut slopes along the roadway. If it is necessary to transition slopes, the transitions should be made such that the maximum longitudinal slope (with regard to the grade line) along the roadside does not exceed 30:1. The 30:1 slope should be based on the sideslope elevation differences at the edge of each respective clear zone.

For example, a transition may be needed from a 6:1 fill slope to a 6:1 cut slope at a bridge overpass. This should be accomplished over a distance calculated as follows:

1. Given: Design Speed = 110 km/h and Design Year ADT = 7500 VPD.

2. Distance to shoulder slope break = 3.3 m from edge of traveled way

3. Elevation differential from slope break for 6:1 fill slope @10.5 m =

$$\frac{10.5 - 3.3}{6} = 1.20 \text{ m}$$

4. Elevation differential from slope break for 6:1 cut slope @10.5 m =

$$\frac{10.5 - 3.3}{6} = 1.20 \text{ m}$$

5. Change in elevation along roadside at clear zone limits = 1.20 m + 1.20 = 2.40 m.

6. Transition distance @30:1 longitudinal slope = 2.40 x 30 = 72 m.

Therefore, the transition from the 6:1 fill slope to the 6:1 cut slope should occur over approximately a 72-m distance along the roadway.

49-3.05(04) Pier Protection (Outside Shoulder)

Pier protection guardrail lengths for the right shoulder of a multi-lane divided highway and both shoulders of a 2-lane, 2-way highway are based on the clear zone and the lateral location of the pier end(s) relative to the clear zone. Collision walls are required where the traffic side face of the pier is not completely protected by guardrail and at those locations where there is a gap between adjacent (in-line) piers which is not completely protected by guardrail. Depending on the lateral locations of the pier and the guardrail, the guardrail should either be fastened to the end of the pier or placed in front of the pier. The location/attachment is discussed below.

The additional guardrail length required to protect other hazards in the area of the structure, such as the slope wall, the bridge cone and the drainage culvert under the slope wall are computed separately.

When the conditions indicated below require calculations to determine the pier protection guardrail length, the calculation should consider all hazards adjacent to the pier end. These requirements apply to piers for single and twin (side-by-side) overhead structures spanning 2-lane, 2-way roadways and multi-lane roadways and tandem (end-to-end) overhead structures spanning multi-lane roadways. The required length of pier protection guardrail is determined in accordance with the following:

1. Pier Located ≤4.9 m from Edge of Travel Lane. The pier protection guardrail must be attached to the upstream traffic end of the pier with a Type GP transition. The minimum required length of guardrail, including the guardrail transition, is shown in Figure 49-3L, Length-of-Need Requirements for Pier Protection, and described below.

    a. If the pier end is located outside of the clear zone and the design speed ≥ 80 km/h, then the minimum required length is 30 m.

    b. If the pier end is located outside of the clear zone and the design speed < 80 km/h, then the minimum required length is 15 m.

    c. If the pier end is located inside the clear zone and the design speed ≥ 80 km/h, then the required length will be based on the clear zone requirements for the roadway. The length of need is calculated using the equations in Section 49-5.02 and the clear zone values from Figure 49-2A, Clear Zone Distances (m) (New Construction/Reconstruction). The calculated lengths are rounded up to the nearest whole multiple of 1.905 m. The amount of guardrail required will be the greater of the calculated rounded length or 30 m.

    d. If the pier end is located inside the clear zone and the design speed is <80 km/h, then the required length will be based on the clear zone requirements for the roadway. The length of need should be calculated using the equations in Section 49-5.02 and the clear zone values from Figure 49-2A. The calculated lengths are rounded up to the nearest whole multiple of 1.905 m. The amount of guardrail required should be the greater of the calculated rounded length or 15 m.

2. Pier Located > 4.9 m from the Edge of Travel Lane. The length of guardrail in advance of the pier(s) is determined in the same manner as that required for all extended hazards along the roadway. The pier protection guardrail should be located between the pier and the edge of travel lane and as far away from the edge of travel lane as feasible.

    The lateral extent of the pier foundation will usually dictate how close the guardrail posts can be driven to the pier face. The guardrail should be located such that the clearance from the guardrail face to the pier face ≥ 1.30 m and the clearance from the guardrail face to the pavement side edge of the pier foundation ≥ 0.53 m. These clearances are needed to permit the guardrail to deflect upon impact without impacting either the pier face or the foundation and to permit the driving of the post. If the clearance from the guardrail face to the pier face < 1.30 m, then the guardrail post spacing must be reduced in accordance with Figure 49-4A, Guardrail Clearances. If the clearance from the guardrail face to the pier face < 0.84 m or the clearance from the guardrail face to the pavement side edge of the pier foundation < 0.53 m, then the guardrail should be installed in accordance with Item 1.

The required length of guardrail is shown in Figure 49-3L, Length-of-Need Requirements for Pier Protection, and is described in Item 1 above. The length of guardrail along the face of the outside shoulder pier/frame bent on multi-lane divided roadways should be sufficient to continuously cover the full length of the pier plus 7.6 m. For twin (in-line) piers, this length will also include the gap between the piers.

49-3.05(05) Pier Protection (Median)

The type of protection required for piers and frame bents located anywhere in the median of a multi-lane roadway is determined by the configuration of the overhead structure(s). The possible overhead structure configurations are single, twin (side-by-side) and tandem (in-line). Collision walls are required where the traffic side face of the frame bent is not completely protected by guardrail and also at those locations where there is a gap between adjacent (in-line) piers/frame bents which is not completely protected by guardrail. The required pier protection is determined as follows and is summarized in Figure 49-3M, Pier Protection Guardrail Requirements.

1. Single Overhead Structure Piers and Frame Bents. The protection required is based on the clearance from the side of the pier/frame bent to the median edge of the travel lane. Where this clearance ≥7.6 m, both ends of the pier/frame bent should be protected by an impact attenuator Type MP. Where this clearance < 7.6 m, both ends of the pier/frame bent should be protected with an impact attenuator type R2.

2. Twin (Side-By-Side) Overhead Structure Piers and Frame Bents. A collision wall should be installed in the gap between the twin piers/frame bents. The protection required at the outermost ends of the pier/frame bent will be based on the clearance from the side of the pier/frame bent to the median edge of the travel lane. Where this clearance ≥7.6 m, both ends of the pier/frame bent should be protected by an impact attenuator Type MP. Where this clearance <7.6 m, both ends of the pier/frame bent should be protected with an impact attenuator type R2.

3. Tandem (In-Line) Overhead Structure Piers and Frame Bents. Due to the bridge cone location behind the median side piers/frame bents for this type of overhead structure, the pier protection for this situation will be the same as that required for outside shoulder locations presented in Section 49-3.05(04).

49-3.06 Signing, Lighting and Signalization

The following presents the roadside safety criteria for signs, lighting and signal poles within the clear zone.

1. Exit Gore Signs. Exit gore signs should be placed in all gore areas on the Interstate system as shown on Figure 49-3N, Sign Gore Treatment.

2. Breakaway Supports. Any substantial remains of breakaway sign and lighting supports or of a guardrail end treatment post, which will remain after the unit has been struck, will have a maximum projection of 100 mm (see Figure 49-3 O, Breakaway Support Stub Clearance Diagram and Light Standard Treatment).

3. Signs. All supports for ground mounted signs will be breakaway or yielding (except those behind an adequate length of guardrail to protect errant motorists from the sign support and at locations with sidewalks). New sign supports behind guardrail should have adequate clearance to the back of the guardrail post to provide for the guardrail dynamic deflection (see Section 49-4.0).

4. Lighting. All conventional light standards will be breakaway except at locations with sidewalks. The location of all breakaway light standards (except those shielded by guardrail) should not be placed in areas where the opportunity exists for them to be struck more than 230 mm above the normal point of vehicular bumper impact. Normal bumper height is 460 mm. To avoid light standards being struck at an improper height, they should be placed, and the area around them graded, as follows:

    a. Fill Slopes Flatter than 6:1. No restrictions on locations, nor is any special grading required. Generally, light standards should be placed 6.0 m from the edge of the travel lane or 3.0 m from the edge of shoulder.

    b. Fill Slopes from 5:1 to 6:1. Follow grading plans as shown in the INDOT *Standard Drawings*. Generally, light standards should be placed 6.0 m from the edge of the travel lane or 3.0 m from the edge of shoulder.

    c. Fill Slopes 4:1 or Steeper. Light standards should be offset 1.0 m from the edge of shoulder or 3.6 m from the edge of the travel lane, whichever is greater. Grading should be provided as shown in Figure 49-3 O.

    d. All Cut Slopes. Follow grading plans as shown in the INDOT *Standard Drawings*.

    Existing breakaway light standards should be evaluated to determine if it is necessary to relocate them, regrade around the bases, or upgrade the breakaway mechanism to current AASHTO standards. The determination of the extent of work necessary on existing breakaway light standards involves a review of numerous variables. Therefore, this determination must be made by INDOT's Highway Lighting Engineer. If Federal-aid funds will be used for construction and the project is on the National Highway System and is not exempt from FHWA oversight, the FHWA should also be consulted.

5. <u>High Mast Lighting</u>. High mast lighting should be placed to provide a desirable clear zone of 25 m. The minimum clear zone distance will be the roadway clear zone through the area where the high mast lighting is located.

6. <u>Traffic Signals</u>. Traffic signal supports on new construction and reconstruction projects should be placed to provide the roadway clear zone through the area where the traffic signal supports are located. However, the following exceptions will apply:

   a. Channelized Islands. Installation of signal supports in channelizing islands should be avoided. However, if a signal support must be located in a channelizing island, a minimum clearance of 9.0 m should be provided from all travel lanes (including turn lanes) in rural areas and in urban areas where the posted speed is greater than 70 km/h. In urban areas where the island is bordered by barrier curb and the posted speed is 70 km/h or less, a minimum clearance of 3.0 m should be provided from all travel lanes (including turn lanes).

   b. Non-Curbed Facilities (Posted Speeds ≥ 80 km/h and ADT > 1500). Where conflicts exist such that the placement of the signal supports outside of the clear zone is impractical (e.g., conflicts with buried or utility cables), the signal supports should be located at least 3.0 m beyond the outside edge of the shoulder.

   c. Non-Curbed Facilities (Posted Speeds < 80 km/h or ADT ≤ 1500). Where conflicts exist such that the placement of the signal supports outside of the clear zone is impractical (e.g., conflicts with buried or utility cables), the signal supports should be located at least 2.0 m beyond the outside edge of the shoulder.

7. <u>Large Signs</u>. Large signs (over 4.5 m^2 in area) on slipbase breakaway supports should not be placed in areas where the opportunity exists for them to be struck more than 230 mm above the normal point of vehicular bumper impact. Normal bumper height is 460 mm. To avoid signs being struck at an improper height, they should be placed as follows:

   a. Fill Slopes Flatter than 4:1. Signs should be located a minimum of 9.0 m from the edge of the travel lane to the nearest edge of the sign.

b. Fill Slopes 4:1 or Steeper. Nearest sign edges should be located 1.8 m from the edge of shoulder or 3.6 m from the edge of the travel lane, whichever is greater.

8. Roadside Appurtenances. Roadside appurtenances such as large breakaway sign or lighting supports should not be located in or near the flow line of ditches. If these supports are placed on a backslope, they should be offset at least 3.0 m up the slope from the bottom of the ditch.

49-3.07 Miscellaneous Grading

The designer should review the following grading considerations.

1. Gore Areas. Gore areas should be graded with a maximum slope of 10:1 parallel to the roadway.

2. Median Cross Slopes. For reconstruction projects, median cross slopes should be 4:1 maximum, but desirably 6:1 or flatter. For median cross slopes on new facilities, see the INDOT *Standard Drawings*.

3. Shoulder Wedge. On reconstruction projects a wedge on the outside and inside shoulders should be constructed as shown on Figure 49-3P, Shoulder Wedges.

4. Rock Cuts. As indicated in Section 49-3.01(02), rough rock cuts located within the clear zone may be considered a roadside hazard. The following will apply to their treatment:

    a. Hazard Identification. There is no precise method to determine whether or not a rock cut is sufficiently "ragged" to be considered a roadside hazard. This will be a judgment decision based on a case-by-case evaluation.

    b. Debris. A roadside hazard may be identified based on known or potential occurrences of rock debris encroaching onto the roadway.

    c. Barrier Warrants. If the rock cut or rock debris is within the clear zone, a barrier may be warranted.

    d. Barrier Type. Where a barrier is used, a full-section concrete median barrier will typically be used.

49-4.0 ROADSIDE BARRIERS

49-4.01 Barrier Types

Steel heavy post W-beam guardrail, concrete barrier and concrete safety shape bridge rail are the barrier types generally used by the Department. Figure 49-5A, Barrier Deflections, provides the deflection distances for these barriers based on post spacings. The desired distance from the face of the guardrail to the shoulder breakpoint is 1.0 m. In restricted conditions, this may be reduced to 0.43 m. The specific types of roadside barriers used by the Department are as follows:

1. W-beam Guardrail @ 1.905 m. This guardrail is used when the clearance between the guardrail face and the fixed object being shielded ≥ 1.30 m, and the clearance from the guardrail face to the top of the embankment slope ≥ 1.00 m (0.57 m measured from the back of the post).

2. W-beam Guardrail @ 0.955 m. This guardrail is used where the clearance between the guardrail face and the fixed object being shielded ≥ 1.00 m but < 1.30 m, and the clearance from the guardrail face to the top of the embankment slope ≥ 1.00 m (0.57 m measured from the back of the post).

3. W-beam Guardrail @ 0.475 m. This guardrail is used when the clearance between the guardrail face and the fixed object being shielded ≥ 0.84 m but < 1.00 m, and the clearance from the guardrail face to the top of the embankment slope ≥ 1.00 m (0.57 m measured from the back of the post).

4. Double-Faced W-beam Guardrail @ 1.905 m. This guardrail is used as a median side bridge approach guardrail for twin structures, in lieu of Guardrail Class H.

5. Concrete Median Barrier (CMB). The CMB should be considered on the roadside to shield rigid objects where no deflection distance is available. The CMB is typically used on urban freeways wherever a barrier is required. If a rigid object is not continuous (e.g., bridge piers), the designer may use a half-section CMB. To provide the necessary lateral support, backfill should be provided behind the half-section CMB, or the CMB should be tied to a concrete surface with reinforcing steel at its base. If this is not practical, use the full-section CMB.

49-4.02 Existing Guardrail Type B

If existing guardrail type B will be retained on a project, the following should be checked.

1. A W-beam back-up plate is required at all W-beam to blockout connections where the W-beam is not lapped.

2. The height of rail should be a minimum of 685 mm with a maximum height of 760 mm as

measured from the top of the W-beam to the ground surface at the face of rail.

3. A rub rail must also be used, even those runs with a radius of 15 m or less.

4. The flat plate washers (75 mm x 45 mm) should be eliminated from under the head of the bolt holding the W-beam to the blockout except where washers are needed to transmit the forces in the W-beam to the anchor posts to obtain end anchorage. For example, if both ends of a guardrail run have positive anchorage at a bridge pier or through a buried end, all of the 75 mm x 45 mm plate washers should be eliminated except those in the transition. However, if the guardrail run ends without a positive connection, anchorage would have to be achieved through the last 5 posts and the washers must be left on these posts.

5. Grading at the location of the guardrail should be in accordance with Section 49-5.01.

6. It is considered safer for an errant vehicle to traverse an embankment slope as steep as 3:1 at any height than it is for the vehicle to impact a traffic barrier which would shield that slope (see Section 49-4.04). Therefore, on reconstruction projects, it may be necessary to remove portions of existing guardrail to conform to the concept that guardrail should be provided only where clearly warranted. However, on slopes steeper than 4:1, the clear runout area shown in Figure 49-2F, Clear Zone Application for Fill Slopes (Non-Recoverable Slope), must be provided at the toe of slope.

49-4.03 Guardrail Selection

The determining factors in the selection of a type of guardrail are the clearances from the guardrail to the hazard and from the guardrail to the top of the embankment slope. They are based on distances from the face of the guardrail, considering the rail/block out/post "thickness" and the rail deflection properties.

49-4.04 Guardrail Warrants for Embankments

Figures 49-4B, 49-4C, 49-4D, 49-4E, 49-4F, 49-4G, and 49-4G_1 present the Department's criteria for placement of traffic barriers on embankments for design speeds of 60, 70, 80, 90, 100, and 110 km/h, and multi-lane divided and undivided roadways, respectively. Although these figures were developed using 3.6-m lanes and 3.0 to 3.6-m shoulders, they can be used for any lane and shoulder widths. Guardrail for embankments is generally not warranted on facilities with design speeds of 50 km/h or less. Slope-height combinations which fall on or below the curve do not warrant shielding. To adjust for horizontal curvature and grade, use the factors shown in Figure 49-9D, Grade Traffic Adjustment Factor (K_g) and Curvature Traffic Adjustment (Factor (K_c). The following example illustrates how to use these embankment warrant figures.

* * * * * * * * *

Example 49-4.1

Given: 2-lane, 2-way highway
Design Speed = 90 km/h
Design Year ADT = 3000
Tangent Section
Grade = 2%
Foreslope = 2.0:1
Fill Height = 3.0 m

Problem: Determine if guardrail is warranted for the embankment.

Solution: Using Figure 49-4E, it can be determined that guardrail is not warranted based on the embankment. However, the designer should consider the need for guardrail based on other factors (e.g., nearby hazards, accident history).

Example 49-4.2

Given: Same highway section as discussed in Example 49-4.1, but with a horizontal radius of 250 m, the embankment of concern on the outside of the curve and a fill height of 3.0 m.

Problem: Determine if guardrail is warranted for the embankment.

Solution:

1. The Design Year ADT first needs to be adjusted by horizontal curvature factor:

   $K_c = 4.0$ from Figure 49-9D

   Corrected Design Year ADT = 3,000 x 4.0 = 12,000

2. Using Figure 49-4E, it can be determined that guardrail is now warranted based on the embankment height. Section 49-5.0 discusses the appropriate location for installing guardrail.

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49-4.05 Median Barriers

49-4.05(01) Warrants

Figure 49-4H, Median Barrier Warrants, provides the warranting criteria for median barriers on freeways and other multi-lane divided highways which have relatively flat, unobstructed medians. As indicated in Figure 49-4H, median barriers are warranted for combinations of 20 year projected ADT and median widths that fall within the crosshatched area. At low 20 year projected ADTs the probability of a vehicle crossing the median is relatively low. Likewise, for relatively wide medians, the probability of a vehicle crossing the median is relatively low. These conditions are reflected by the shaded area under the curve. For 20 year projected ADTs less than 20,000 and median widths below the warranting curve and for median widths greater than 9.0 m and below the warranting curve, median barrier use is optional.

Figure 49-4H should be used where barriers can be installed with breaks in the barrier 1.5-km or more apart and the design speed is 80 km/h or greater. Access-controlled facilities, where long runs withoutbreaks in the barrier are possible and cross-median accidents are a problem, may use this Figure. If breaks in the median barrier would, on the average, be less than 1.5-km apart, a median barrier should generally not be installed because of the larger number of barrier end treatments required. The hazard created by the end treatments are generally greater than the benefits derived from a median barrier.

49-4.05(02) Design

The following applies to the design of median barriers:

1. Type. Where a median barrier is warranted in narrow medians, Department policy is to only use a concrete median barrier (CMB). The CMB is a rigid system which will rarely deflect upon impact. Either an 840-mm common height, or 1145-mm truck height, CMB will typically be used. See Section 49-9.0 for the selection procedures for the CMB.

   A modified CMB may be necessary where the median barrier must accommodate a fixed object in the median (e.g., bridge piers, sign supports). For design details, see the INDOT *Standard Drawings*.

2. Median Slopes. Median slopes in front of a median barrier should be 20:1 or flatter. Where a median barrier is warranted, it should be placed such that its effectiveness is not diminished by the severity of the median slopes. This may, in some cases, result in the placement of a median barrier along either or both inside shoulders instead of a single barrier along the center of the median.

3. Superelevated Section. Where a median barrier is located on the high side of a superelevated section, its vertical axis of symmetry should be at 90º to the pavement

surface. On the low side of a curve, the axis of symmetry can be either vertical, or at 90º to the pavement surface. See Section 43-3.0 for more information on superelevation development with a CMB section.

4. Barrier-Mounted Obstacles. If trucks or buses impact the CMB, their high center of gravity may result in a vehicular roll angle which may result in the truck or bus impacting obstacles on top of the CMB (e.g., luminaire supports). If practical, these devices should be moved to the outside or additional distance provided between the barrier and obstacle (e.g., bridge piers).

5. Terminal Treatments. As with roadside barrier terminals, CMB terminals also present a potential roadside hazard for run-off-the-road vehicles. Therefore, careful consideration must be given to the selection and placement of the terminal end. For the terminal ends of concrete median barriers, impact attenuators are typically used. See Section 49-6.0 for information on impact attenuators.

49-4.05(03) Glare Screens

Headlight glare from opposing traffic can be bothersome and distracting. Glare screens can be used in combination with median barriers to eliminate the problem. INDOT has not adopted specific warrants for the use of glare screens. The typical application, however, is on urban freeways with narrow medians and high traffic volumes. Another application is between on/off ramps at interchanges where the two ramps adjoin each other. Here, the sharp radii or curvature and the narrow separation may make headlight glare especially bothersome. The designer should consider the use of glare screens at these sites. A key element warranting their use is the number of public complaints received for a particular highway section.

The designer should evaluate the following design criteria for glare screens.

1. Cutoff Angle. Glare screens should be designed for a cutoff angle of 20º. This is the angle between the median centerline and the line of sight between two vehicles traveling in opposite directions. See Figure 49-4 I, Cutoff Angle for Glare Screens. The glare screen should be designed to block the headlights of oncoming vehicles up to the 20º cutoff angle. On horizontal curves, the design cutoff angle should be increased to allow for the effect of curvature on headlight direction. The criteria are as follows:

$$\text{Cutoff Angle (in degrees)} = 20 + \frac{1746.8}{R}$$

Where R = horizontal radius (m).

2. Horizontal Sight Distance. Glare screens may reduce the available horizontal sight distance. For curves to the left, the designer will need to check the middle ordinate to determine if adequate stopping sight distance will be available. See Section 43-4.0.

3. Sag Vertical Curves. When determining the necessary glare screen height, the designer does not need to consider the effect of sag vertical curvature.

4. Height of Eye. The driver's eye height is 1080 mm.

5. Glare Screen Height. To determine the appropriate height of the glare screen, the designer should review NCHRP Synthesis 66 *Glare Screen Guidelines*.

49-5.0 ROADSIDE BARRIER APPLICATIONS

Some of the major factors to consider in the lateral placement of a roadside barrier include the following:

1. clearance between barrier and hazard being shielded to allow for deflection of the barrier;

2. effects of terrain between the edge of the traveled way and the barrier on the errant vehicle's trajectory;

3. probability of impact with barrier as a function of its distance off the traveled way;

4. flare rate and length of need of transitions and approach barriers; and

5. the need to offset guardrail and CMB from the edge of shoulder so that the full shoulder width can be used. On new construction, the desirable guardrail offset is 0.6 m from the effective usable shoulder width. The minimum guardrail offset is 0.3 m from the effective usable shoulder width. On reconstruction projects, the desirable guardrail offset is 0.6 m from the effective usable shoulder width. The minimum guardrail offset is 0 m from the effective usable shoulder width.

49-5.01 Lateral Placement

49-5.01(01) Barrier Offset

A roadside barrier should be placed as far from the traveled way as conditions permit, thereby minimizing the probability of impact with the barrier. The roadside barrier should be placed beyond the shyline offset; see Section 49-5.02(01).

The designer should evaluate the practicality of offsetting the guardrail or CMB more than 0.6 m beyond the edge of the required shoulder width. This assessment must include a comparison of the additional costs of all items such as benching, borrow and grading needed to construct the flat slopes required to install barrier on the embankment, against the reduced cost of installation and maintenance of the lesser amount of barrier which would be required by locating it farther from the roadway. This assessment should also consider the location's accident history and the area's maintenance records regarding the repair of nuisance impacts.

49-5.01(02) Shoulder Section

On INDOT routes, the outside shoulder is paved to the face of guardrail if the face of the rail is located 4.5 m or less from the edge of the travel lane. On local public agency projects, the shoulder section at guardrail locations may be designed as follows:

1. Where the face of the guardrail is ≤0.6 m from the outside edge of the required shoulder width, the shoulder will have the same pavement section to the face of the guardrail as that specified for the required shoulder width.

2. Where the face of the guardrail is >0.6 m from the outside edge of the required shoulder width, the shoulder should only be paved to the required shoulder width.

49-5.01(03) Barrier Deflection

If the distance between the face of guardrail and the face of an isolated hazard is less than the dynamic deflection distance as shown in Figure 49-5A, Barrier Deflections, it will be necessary to reduce the post spacing to obtain a dynamic deflection distance that is less than the clearance between the face of guardrail post and the face of object. If this is not practical, either the object or the guardrail should be moved to provide adequate deflection distance. It should be noted that the CMB does not deflect.
The deflection distances for thrie-beam guardrail are shown, but they should only be used at problem or special locations.

The deflection distances for type B guardrail are given so that the designer can analyze existing installations to determine whether or not existing deflection distances are sufficient.

49-5.01(04) Guardrail and Embankment Slopes

A roadside barrier should not be placed on an embankment if the slope of the embankment is steeper than 20:1 desirable, 10:1 maximum. Also, a barrier should not be placed on an embankment if the difference between the shoulder slope rate and the side slope rate located in front of the guardrail is greater than approximately 8 percent. This will minimize the probability of an errant vehicle vaulting over the barrier (see Figure 49-5B, Guardrail and Embankment Slopes).

49-5.01(05) Guardrail and Curbs

Curbs in front of guardrail may cause an errant vehicle to vault over or break through the rail. However, there has been very little research on which to recommend curb geometry or placement in the vicinity of a traffic barrier. For this reason, the best practice is to avoid using curbs in the vicinity of guardrail. If a curb is essential for drainage, its effect can be minimized by using a maximum curb height of 100 mm and placing it so that the face of the curb is at or behind the face of the guardrail. Therefore, the guardrail post must be driven immediately behind the back of curb.

In urban situations, the guardrail-curb combination should be offset at least the shy line distance from the edge of the travel lane. This offset may either be continuous (curb with or without guardrail) or variable as shown in Figure 49-5C, Guardrail Placement (With Curbs). A continuous offset should be used if there are numerous separate runs of guardrail along a route to provide a uniform curb line offset. Thrie-beam guardrail should be used instead of the standard W-beam guardrail where curbs and sidewalks approach a bridge rail.

Where a guardrail will be installed in the vicinity of an existing curb, the curb should be removed unless the guardrail can be placed as discussed above.

49-5.02 Barrier Length of Need

Figure 49-5D, Barrier Length of Need, illustrates the total length of need of a barrier, which is based on the equation as follows:

$$L_{TOTAL} = L_{ADVANCE} + L_{HAZARD} + L_{OPPOSING}$$

Where:

$L_{ADVANCE}$ = The length of need in advance of the hazard
L_{HAZARD} = The length of the hazard itself
$L_{OPPOSING}$ = The length of the trailing end or length needed to protect traffic in opposing lanes.

49-5.02(01) Barrier Length Needed in Advance of Hazard

Figure 49-5E, Barrier Length of Need (Advance of Hazard), illustrates the variables in the layout of an approach barrier to shield an area of concern for adjacent traffic. Generally, roadside barriers should be installed parallel to the roadway; however, flared installations may be appropriate in specific cases such as where the guardrail end is buried in the backslope. Figure 49-5F presents the runout length (L_R) and shy line offset (L_S) as a function of design year ADT and design speed. Figure 49-5G provides the flare rate (a:b), relative to the shy line. The shy line offset is defined as the distance beyond which a roadside obstacle will not be perceived as a threat by a driver. The roadside barrier should be placed beyond the shy line offset except as described in Section 55-5.04(02) for 3R projects. The following procedures are used to determine the barrier length of need.

1. Graphical Solution (Tangents & Inside Horizontal Curves). One method of determining the length of need is to scale the barrier layout directly on the highway plan sheets as shown on Figure 49-5H, Example of Barrier Design (Bridge Approach). First, the runout length (L_R) is selected from Figure 49-5F. Then, the lateral distance to be protected is determined by calculating the clear zone (L_C) and comparing it to the lateral distance from the edge of travel lane to the outside edge of the hazard (L_H). Generally, the lesser of these two distances (L_C and L_H) is the value used to calculate the length of need, although the designer may choose to protect a wider area. Next, the runout length (L_R) and the lateral distance to be protected are scaled on the drawing along the edge of the travel lane, and a line is drawn between the lateral point farthest from the edge of the travel lane and the end of the runout length farthest from the hazard. This line simulates the vehicular runout path. To shield the hazard, the barrier installation must intersect this line. The barrier may be either flared or parallel to the roadway as dictated by site conditions.

2. Graphical Solution (Outside Horizontal Curves). For barrier designs on the outside of horizontal curves, the graphical solution should be used. Note that the barrier length of need is determined by scaling its intercept with the tangential runout path of an encroaching vehicle rather than using the approach runout length, L_R. This is illustrated in Figure 49-5 I, Example of Barrier Design (Fixed Object on Horizontal Curve). However, if the runout length measured along the edge of the driving lane is shorter than the distance to the tangential runout path intercept, the shorter distance should be used.

3. Mathematical Solution (Tangent Sections Only). The required length of need may be calculated using the following formulas:

   For a flared run of guardrail:

$$X = \frac{L_H + (b/a)(L_1) - L_2}{b/a + (L_H)/(L_R)} \qquad \text{(Equation 49-5.1)}$$

$$Y = L_H - \frac{(L_H)}{(L_R)}(X) \qquad \text{(Equation 49-5.2)}$$

For a parallel installation of guardrail:

Where:

X = length of need in advance of the hazard
Y = lateral offset to beginning of length of need on a flared run of guardrail

Other variables are defined in Figure 49-5E, Barrier Length of Need (Advance of Hazard).

4. Minimum Length of Rail. If the design speed is 80 km/h or greater, the required length of guardrail in advance of the hazard will be the greater of the calculated length or 30 m where GRET type I is used. Otherwise, such length will be the greater of the calculated length or 15 m.

   If the design speed is 70 km/h or less, the required length of guardrail in advance of the hazard will be the greater of the calculated length or 15 m.

49-5.02(02) Barrier Length Needed for Opposing Traffic

Figure 49-5J illustrates the layout variables of an approach barrier for opposing traffic. The length of need and the end of the barrier are determined in the same manner as for adjacent traffic, but all lateral dimensions are measured from the edge of the travel lane of the opposing traffic (e.g., from the centerline for a 2-lane roadway). If there is a 2-way divided roadway, the edge of the travel lane for the opposing traffic would be the edge of the driving lane on the median side. If guardrail is necessary to protect traffic in the opposing lanes, the minimum length of guardrail is determined as follows:

1. If the design speed is 80 km/h or greater, the required length of guardrail in advance of the hazard for opposing traffic will be the greater of the calculated length or 30 m where GRET type I is used. Otherwise, such length will be the greater of the calculated length or 15 m.

$$X = \frac{L_R(L_H - L_2)}{(L_H)} \qquad \text{(Equation 49-5.3)}$$

2. If the design speed is 70 km/h or less, the required length of guardrail in advance of the hazard for opposing traffic will be the greater of the calculated length or 15 m.

There are three ranges of clear zone width (L_C) that deserve special attention for an approach barrier for opposing traffic. In analyzing these situations, the designer should use good judgement in determining the type of treatment for a barrier or hazard when the barrier or hazard is just outside of the clear zone. These ranges are as follows:

1. If the barrier is beyond the appropriate clear zone, no additional barrier is required. However, a crashworthy end treatment should be considered based upon ADT, distance outside the clear zone and roadway geometrics.

2. If the barrier is within the appropriate clear zone but the hazard is beyond it, no additional barrier is required but a crashworthy end treatment should be used.

3. If the hazard extends well beyond the appropriate clear zone (e.g., a river), the designer may choose to shield only that portion which lies within the clear zone, by setting L_H equal to L_C.

49-5.02(03) Length of Need Beyond Hazard (Divided Highways)

Figure 49-5K, Guardrail Length Beyond Hazard (Divided Highways), illustrates the procedure for determining the length of need beyond the hazard on divided highways.

49-5.03 W-Beam Guardrail Over Low-Fill Culverts

Large drainage structures are defined as those with clear spans of at least 1675 mm, as measured parallel to the roadway centerline, and three-sided culverts. For such structure ends within the clear zone which are costly to extend and whose end sections cannot be made traversable, shielding with guardrail should be provided to protect errant motorists from colliding with a structure end. If the structure end is outside the clear zone, guardrail should be placed to protect the errant motorist from the structure end.

If there is inadequate cover over the structure to support the guardrail posts, it will be necessary to use the details for guardrail installations over low-fill structures as shown in the INDOT *Standard Drawings*. In such situations, full embedment of the guardrail posts is often impractical. The designer should also show on the plans where the various types of standard or modified posts are to be used.

In certain situations, steel or concrete bridge railing in accordance with National Cooperative Highway Research Program Report (NCHRP) 350 criteria also may be required over low-fill structures where modified guardrail posts cannot be utilized. An appropriate guardrail to bridge railing transition should be used.

The nested guardrail configuration shown on the INDOT *Standard Drawings* should be used where there is inadequate cover for driving full-length guardrail posts. The configuration may be used within a longer run of W-beam guardrail, or may be used alone, depending on the length of guardrail need. This configuration has been crash tested in accordance with NCHRP 350 requirements, and approved for use by the FHWA on the National Highway System.

The configuration may only be used as one complete 30.48 m unit. The designer should determine the number of modified posts, if they are required, to determine the pay quantity. The designer should also determine the end treatment requirements.

The length of need for guardrail in advance of the structure or area of concern should be determined as required by Section 49-5.02. If nested W-beam guardrail is used over the structure and is not sufficient for the calculated length of need, additional standard W-beam guardrail should be provided to satisfy the length-of-need requirements preceding the nested W-beam guardrail installation as shown on the INDOT *Standard Drawings.* Likewise, if there is a need for a standard W-beam guardrail beyond the nested W-beam guardrail installation, the standard W-beam guardrail (minimum length 7.62 m) should be connected to the outgoing end of the nested W-beam guardrail installation in lieu of the cable terminal anchor system.

On all projects including 3R, where W-beam guardrail is used to shield a structure, the following procedure should be used for the various combinations of overall structure width (W, mm), and depth of cover (C, mm) over the structure. The overall structure width of a large drainage structure is defined as the width out to out of structure parallel to the roadway centerline for skewed or perpendicular structures.

49-5.03(01) Longitudinal Placement of Guardrail over Large Drainage Structures

1. $W \leq 7400$ and $C < 1250$. Use nested guardrail including a 7620 mm span over the structure as shown on the INDOT *Standard Drawings.*

2. $7400 < W \leq 18600$ and $500 \leq C < 1250$. Use nested guardrail including a 7620 mm span over the structure, and modified posts for the nested guardrail adjacent to the 7620 mm span as shown on the INDOT *Standard Drawings*. The modified posts should be inserted into steel tubes, which are embedded into concrete bases. The concrete post bases should not be attached to the structure. The modified posts with concrete bases should only be used over the structure.

3. W = Any Structure Width and $1250 \leq C < 1550$. Use standard W-beam guardrail with 1830 mm length posts at 1905 mm spacing over the structure, and 2130 mm length posts at 1905 mm spacing preceding and beyond the structure.

4. W = Any Structure Width and $C \geq 1550$. Use standard W-beam guardrail with 2130 mm length posts at 1905 mm spacing.

49-5.03(02) Lateral Guardrail Placement for Large Drainage Structures on Projects on New Alignments Excluding 3R Projects

For these projects it is desirable to perpetuate as much of the clear zone as practical through a structure location. Where sufficient right of way will be acquired to provide the required clear zone, the guardrail systems described in Section 49-5.03(01) may be installed near the clear zone limits. This is to shield the structure ends which are located within the clear zone, thus maintaining most of the clear zone required over the structure. However, where these guardrail systems are utilized near the edge of the clear zone, these systems should not be connected to any other existing or proposed guardrail that is located nearer to the pavement.

49-5.03(03) Lateral Guardrail Placement for Large Drainage Structures on Projects on Existing Alignments and 3R Projects on New Alignments

Many of these projects may not have sufficient right of way to perpetuate the clear zone through the structure location. In these situations, the guardrail should be installed at an offset of up to 0.6 m from the edge of shoulder. These situations will require standard or nested W-beam guardrail depending on the depth of cover available over the structure.

49-5.03(04) Bridge Railing Over Structures With W > 7400 mm and C < 500 mm

If the structure has insufficient cover and the clear span is greater than 7400 mm, it may be cost effective to provide a bridge railing over the opening. However, this design requires an appropriate transition from W-beam guardrail to the bridge railing, thereby increasing the installation cost. A special bridge railing design will be required to accommodate specific site conditions. The lateral placement considerations described in B above apply to the bridge railing also.

49-5.03(05) Cable Terminal Anchor System

The cable terminal anchor system may be used at the outgoing end of any W-beam guardrail run that is not exposed to oncoming traffic. It may be used as the equivalent of the W-beam anchorage guardrail ordinarily required 7.62 m beyond the length of need, where space limitations do not permit placement of such a guardrail run.

49-5.03(06) Grading Requirements for Large Drainage Structures

Grading requirements for structures for rural divided highways on new alignment with a design speed of 110 km/h are shown on the INDOT *Standard Drawings*. For other design speeds, similar grading configurations should be designed using appropriate design criteria and dimensions.

Grading requirements for structures for highways on existing alignment with any design speed are also shown in the INDOT *Standard Drawings* for grading requirements at guardrail end treatments.

Guardrail length of need should be based on the project clear zone.

49-5.04 Guardrail End Treatments and Transitions

49-5.04(01) Guardrail End Treatments and Usage

There are four basic types of guardrail end treatments (GRETs). They are as follows:

1. Guardrail End Treatment Type OS. This type of GRET dissipates energy when hit head-on and has the ability to redirect an errant vehicle on one side only, where backside impacts are not anticipated. It is used with single-faced guardrail.

2. Guardrail End Treatment Type MS**.** This type of GRET dissipates energy when hit head-on and has the ability to redirect an errant vehicle on two sides, where backside impacts are anticipated. It is used with double-faced guardrail.

3. Guardrail End Treatment Type I**.** This type of GRET may be used only on local public agency routes and local approaches to INDOT routes where the design year ADT < 1000 regardless of the design speed. Double-faced Guardrail End Treatment Type I may be used in conjunction with double-faced guardrail installations. The Type I Guardrail End Treatment is shown in the INDOT *Standard Drawings*. This guardrail end treatment type shall not be used on the state highway system.

4. Guardrail End Treatment Type II. This type of GRET is used where cut slopes or backslopes above the roadway grade are encountered along the roadside. The Type II GRET is shown in the INDOT *Standard Drawings*. Type II GRET is used to terminate single-faced guardrail in backslopes. This type redirects an errant vehicle on one side only. In some cases it may be necessary to modify the details on the INDOT *Standard Drawings* to adapt to unique conditions. Any deviation from the *Standard Drawings* should be shown on the plans and include the design characteristics relative to guardrail design and embankment slopes as shown in the *Standard Drawings*.

   Where practical, it is desirable to bury the ends of guardrail runs into the backslopes. Important factors to consider when burying guardrail in backslopes are proper guardrail flare, maintaining the proper height of the guardrail, providing proper shoulder, embankment and approach slopes in front of the guardrail and maintaining drainage.

The following lists several design considerations the designer should evaluate in the selection of a Type II GRET:

a. In order to use Type II GRET, a minimum 22.86 m straight run of standard W-beam guardrail which may include a guardrail transition, is required preceding the area of concern (hazard).

b. If this 22.86 m guardrail run is not adequate, extend the guardrail run to shield the hazard.

c. The cut slope or backslope should be located laterally approximately 2 m minimum and 5.25 m maximum from the face of guardrail, at the end of the 22.86 m guardrail run. The designer should ascertain that the backslope extends parallel to the roadway for a sufficient distance to bury the end of the Type II GRET; otherwise, a different type of GRET will be required.

d. The total length of Type II GRET is measured from the end of the WR-beam guardrail run to the last post of the steel post anchor system of the Type II GRET. This buried-in backslope guardrail system is made up of three components as follows:

    (1) The first component is 7.62 m long WR-beam guardrail at the specified ratio a:b, depending upon the project design speed at the specific location.

    (2) The length of the second component which is also WR-beam guardrail varies from 0 to 30.48 m to fit field conditions at the specified ratio a:b, depending upon the project design speed at the specific location.

    (3) The third component is 11.43 m long plus the steel post anchor system at the specified ratio 8:1.

e. For the buried-in backslope guardrail system to be cost effective, the total length of the system should not exceed approximately 50 m beyond the guardrail length of need as determined in Section 49-5.0.

49-5.04(02) Railing Transitions and Usage

The specific railing transitions used by the Department are as follows:

1. Bridge Railings. For concrete bridge railings, the Department uses two transitions. In one, a thrie-beam guardrail element is attached to the bridge railing transition. In the other, a W-beam guardrail element is attached to the bridge railing transition. Each system includes both a guardrail transition and a bridge railing transition. The details are shown in the INDOT *Standard Drawings*. The two transitions will be used for the 840-mm, or common height, F-shape concrete railing; the 1145-mm, or truck height, F-shape concrete railing; the Texas T411, or TX concrete railing; and the side-mounted thrie-beam railings. The general usage is as follows:

    a. Type TGB. This is the preferred transition. It is typically used in all locations, except where an intersecting road or driveway prevents the placement of a proper design. To use the Type TGB, there must be space to place at least 7.62 m of roadside barrier between the curved W-beam guardrail connector terminal system or curved W-beam guardrail system and the beginning of the Type TGB guardrail transition.

    b. Type WGB. This transition type is used where the proximity of an intersecting road or driveway prevents the proper installation of the guardrail Type TGB transition. Note that, where at least one Type WGB transition is required at a bridge, all bridge ends should use the Type WGB transition.

    Details for the two-tubed, curb-mounted bridge railing are shown in the INDOT *Standard Drawings.*

    See Section 61-6.0 for more information on the location and design of transitions used at bridge railings.

2. Guardrail Transition Type GP. This transition is used to connect guardrail to bridge piers and frame bents.

3. Guardrail Transition Type VH. This transition is used to extend existing Guardrail Classes Bs, Ds, Es and Hs when adding new guardrail. This transition involves the vertical adjustment of the first 7.620 m of existing guardrail adjacent to the new guardrail. The adjustment requires the posts in this 7.620-m section to be driven deeper to compensate for the height difference between the two guardrail systems, and it also requires the proper termination of the rub rail. This transition is also used wherever guardrail end treatment Types MS or OS are being connected to an old railing system. To properly specify the required version of this transition, the post spacing of the existing guardrail adjacent to the proposed extension must be known.

49-5.04(03) Design Considerations

The following lists several design considerations the designer should evaluate in the design of end treatments and guardrail transitions:

1. Slopes. All slopes in the area of guardrail end treatments should be graded in accordance with the INDOT *Standard Drawings*.

2. Breakaway Cable Terminals. Breakaway cable terminal end sections will be removed and replaced with a standard end treatment which is suitable for the location.

3. Transitions. Any guardrail transition to bridge piers, bridge rails, etc., will be in accordance with the INDOT *Standard Drawings*.

4. Openings Near Bridges. Occasionally, a driveway or a county road will intersect the highway a short distance from the end of the bridge. Providing openings in the guardrail for these approaches will be accomplished by using the curved W-beam guardrail terminal or connector systems as shown in the INDOT *Standard Drawings*.

5. Guardrail End Treatment Type I. Guardrail end treatment type I is not permitted on the state highway system. All such end treatments should be flared. The embankment in the flared area should be sloped at a 20:1 rate. If the guardrail is already on a taper, it is acceptable to continue the buried end on the same taper line without offsetting it further, provided the minimum 0.6-m offset is obtained.

6. Type OS and MS Guardrail End Treatments. Guardrail end treatments, Type OS and MS, should be installed in alignment with the guardrail if the guardrail run is on a tangent. For curved guardrail runs, construct the Type OS and MS end treatments along a chord of the curve with the beginning and ends of the end treatment having the same offset from the edge of the travel lane (see Figure 49-5 O, Types OS and MS Curved Treatment).

7. Buried W-Beam Guardrail in Backslopes. Where practical, consideration should be given to burying the ends of a guardrail run into the backslope. Important principles to consider when burying guardrail in backslopes are proper guardrail flare, maintaining full design height of guardrail, and providing proper drainage and approach terrain details. In addition, the designer should consider the following:

   a. Flare Rates. It is recommended that the W-beam rail system be flared away from the roadway at a rate no greater than 15:1 until the guardrail passes the clear zone or the center of the ditch, whichever is the greater distance. At that point, it can then be flared back at 8:1. The foreslope in front of the guardrail should be 20:1. A steeper slope, up to a maximum of 10:1, may be used if necessary to allow for ditch grading.

b. Guardrail Height. The design height of guardrail should be maintained across the slope to the point where the guardrail passes over the foreslope/backslope intercept. In areas where this is not practical and if the gap between the ground and the bottom of the W-beam rail is 510 mm or more, it will be necessary to add a W-beam rub rail. The rub rail should be added for 15.240 m downstream and 7.620 m upstream of the area where the gap exceeds the 380-mm normal height. The W-beam rub rail should be terminated behind the last post, similar to that shown for a Type VH transition in the INDOT *Standard Drawings*.

c. Anchors. The end of the guardrail buried in the backslope will be anchored with a W-beam steel post anchor system as shown in the INDOT *Standard Drawings*.

d. Transitions. A foreslope transition zone will be needed to transition from the standard ditch cross-section in the cut section to the 10:1 desirable, 6:1 maximum, foreslope in front of the guardrail. The approach slope to the 20:1 cross slope in front of the guardrail should be a 30:1 maximum longitudinal slope relative to the roadway grade. The ground can then be warped from the standard ditch cross-section to the desired 10:1 foreslope in front of the guardrail. These conditions, if met, should minimize the potential for vehicles to vault over the guardrail or for wheels to snag on the guardrail.

e. Drainage. Where a special ditch section providing the recommended guardrail approach terrain cannot be constructed without blocking flow in the ditch or where the resulting ditch grade is too slight, an acceptable inlet type and an outlet pipe will be required to carry the drainage under the guardrail. Even where an inlet is not needed in the vicinity of the guardrail because of approach terrain requirements, there may be a need for a drainage structure behind the guardrail in the fill section to prevent erosion.

9. <u>Drive Behind</u>. If an errant vehicle penetrates the guardrail end treatment section, the driver should be able to guide his vehicle down the slope without problems. Therefore, a minimum recovery area behind the barrier end treatment must be provided on all projects. This recovery area is depicted in Figure 49-5Q, Clear Recovery Area Behind the Guardrail.

49-5.04(04) Design Procedure

After the design of a roadside barrier is completed, including the appropriate railing transitions and the determination of the barrier length of need in accordance with Section 49-5.0, it is necessary to select the proper guardrail end treatment type (MS, OS, I or II) for the guardrail in accordance with Section 49-5.04(01).

In order to determine the appropriate type of GRET, the following information should be considered:

1. Relationship of Guardrail End Treatment to Traffic. The designer must determine if there will be traffic on one or both sides of the guardrail end treatment. Will the GRET be located beyond the outside shoulder with traffic passing on one side only or will it be in a median, gore, or other location where traffic passes on two sides? If all traffic will pass a GRET only on one side, the GRET will not require redirective capability on more than one side. If traffic will pass the GRET on two sides, it may be necessary for the GRET to be capable of redirecting errant vehicles from two sides.

    a. GRET for Single-Faced Guardrail. For this situation, the GRET must provide redirective capability only on the traffic side. GRET Type OS or Type II should be selected for this situation.

    b. GRET for Double-Faced Guardrail. For this situation, the GRET must provide redirective capabilities on both sides. GRET Type MS should be selected for this situation.

    c. Guardrail End Treatment Along a Local Public Agency Route Where the Design Year ADT is < 1000. For this situation, the GRET Type I may be selected regardless of the design speed. Double-faced Guardrail End Treatment Type I may be used in conjunction with double-faced guardrail. However, GRET Type I shall not be used on the National Highway System.

2. Relationship Between Guardrail End Treatment and Guardrail Length of Need. A 3.81-m portion of the downstream ends of GRETs OS and MS can function as typical guardrail and can be considered as part of the length of need in advance of the obstruction. Therefore, where Types OS and MS are selected as GRETs, the pay length for the guardrail run is equal to the required length of need for the guardrail minus 3.81 m. This reduced pay length is to be reflected in the guardrail length shown on the plans.

49-5.05 Example Guardrail Calculations

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Example 49-5.1

Given: Divided highway structure over stream
Design speed = 110 km/h

Volume = 7000 ADT
Foreslope = 4:1

Problem: Determine the length of the guardrail needed on the shoulder side for the approaching end of the structure.

Solution: (See Figure 49-5R, Guardrail Length of Need – Structure Approach (Example 49-5.1)).Clear Recovery Area Behind the Guardrail.

1. From Figure 49-2A, Transverse Slopes ,determine clear zone (CZ) = 14.0 m.

2. From Figure 49-5F, Design Elements for Barrier Length of Need ,determine runout length (L_R) = 140 m.

3. To find the point of clear zone (CZ), the designer must first determine what is the hazard; in this case, it is the stream and the designer must protect an errant vehicle from the stream.

4. To establish the point of clear zone, the designer must first determine if the CZ is outside the right-of-way. If it is outside the right-of-way, then the right-of-way line becomes the CZ and where it crosses the top of the bank of the stream becomes the point of clear zone.

5. From the point of clear zone, the designer draws a line perpendicular to the edge of the travel lane and calls this point E_P.

6. From the point E_P, scale off distance L_R along the travel lane edge and call this point E_R.

7. From point E_R, to the point of clear zone, draw a line.

8. Draw a line along the face of guardrail parallel to the centerline from the bridge rail to where it crosses the line between E_R and the point of clear zone. This is the length of need for the guardrail for this particular bridge.

9. Add an appropriate end treatment.

<u>Example 49-5.2</u>

Given: 2-lane highway with high fill
Design speed = 100 km/h
Traffic volume = 7000 ADT
Right shoulder width = 3.0 m
Slope in high fill area = 2.5:1

Slope at toe of fill = flat
Tangent
Level Conditions

Problem: Determine the length of guardrail needed to protect the fill slope.

Solution: (See Figure 49-5S)

1. Determine clear zone from Figure 49-2A. CZ = 9.0 m based on flat slope at toe of fill. Therefore, adjusted clear zone = 9.0 - 3.0 shoulder; 6.0 m at toe of slope.

2. Determine runout length from Figure 49-5F; L_R = 125 m.

3. From Figure 49-4F, determine location where guardrail should start. Interpolating between the 6000 ADT and the 12,000 ADT lines, the fill height = 2.7 m.

4. At the point where the fill is 2.7 m high, scale the L_R distance to point E_R.

5. From point E_R to point of clear zone, draw a line.

6. Draw a line along the face of rail parallel to centerline from the point where the fill height is 2.7 m to where it crosses the line, between E_R and the point of clear zone. This is the length of need to shield the driver from the fill height.

7. Add an appropriate end treatment.

8. The trailing end of a run of guardrail is determined in a similar manner, however, the clear zone width is measured from the near edge of the opposing travel lane, see Section 49-5.02(02).

<u>Example 49-5.3</u>

Given: Divided highway with large box culvert within clear zone that cannot be extended (under fill).
Design speed = 110 km/h
Traffic volume = 7000 ADT
Foreslope = 5:1

Problem: Determine the length of guardrail needed to protect the driver from the culvert end.

Solution: (See Figure 49-5T).

1. Determine clear zone from Figure 49-2A; CZ = 11.5 m.

2. Determine runout length from Figure 49-5F; L_R = 140 m.

3. Using the end of the wing on the approaching traffic side of the box culvert, draw a line perpendicular to the edge of the travel lane from the point of a clear zone through the end of the wing to the edge of the travel lane and call this point E_P.

4. From point E_P, scale along the travel lane the distance L_R and call this point E_R.

5. From point E_R to point of clear zone, draw a line.

6. Draw a line along the face of rail parallel to centerline from point E_P to where it crosses the line, between E_R and the point of clear zone. This is the length of need for the guardrail on the approaching traffic side.

7. The trailing end of a run of guardrail for the protection of the box culvert should be extended far enough to protect an errant vehicle from any hazard (in this case, a Type F paved side ditch) when leaving the roadway at a 25º angle and missing the end of the guardrail. When this point has been established, add an additional 7.620 m to establish the strength of the guardrail run.
8. On approaching traffic end, add a Type OS guardrail end treatment and, on the other end, add a curved terminal end.

* * * * * * * * * *

49-6.0 IMPACT ATTENUATORS

49-6.01 Types of Impact Attenuators

The Department uses five types of impact attenuators. They are as follows:

1. Type ED. Type ED impact attenuators are energy dissipation devices.

2. Type R1. Type R1 impact attenuators are energy dissipation devices that have redirective capability on one side.

3. Type R2. Type R2 impact attenuators are energy dissipation devices that have redirective capability on two sides.

4. Type CR. Type CR impact attenuators are also energy dissipation devices that have redirective capability on two sides. These attenuators are used at locations where there are lateral clearance restrictions that make installation and maintenance of the attenuator difficult.

5. Type LS. Type LS impact attenuators are low speed energy dissipation devices that have redirective capability on two sides. These attenuators shall be in accordance with Test Level 1(TL-1) criteria only.

49-6.02 Design

After the design of roadside barriers is performed in accordance with Section 49-5.0, it is necessary to determine whether there are any obstructions located within the clear zone that are not protected. Obstructions that can be protected by extending a proposed barrier a short distance should be protected in that manner. However, impact attenuators should be utilized to protect isolated obstructions.

Figure 49-6A, Impact Attenuator Offset Examples, illustrates common impact attenuator installations. The D1 dimension shown on the figure determines whether an attenuator is warranted and, if so, whether the attenuator requires redirective capability on the side adjacent to the traffic under consideration. The D2 dimension shown on the figure is used to determine whether the attenuator requires redirective capability on its backside.

For obstructions in gore or similar areas, the offset dimension from the edge of the obstruction face to the mainline outside travel lane edge must be compared to the similar measurement between the obstruction and the ramp inside travel lane edge. The smaller of the two offsets is defined to be D1 and the larger offset is considered to be D2.

The attenuator type is determined by using Figure 49-6B, Impact Attenuator Type Determination. The attenuator selection design is based on the appropriate test level for the project design speed of the roadway under consideration. Attenuator Type LS should be selected for a design speed of 50 km/h or lower and the attenuator design should be in accordance with TL-1 criteria. Attenuators for higher design speeds should be in accordance with TL-2 or TL-3 criteria. If the design speed is 70 km/h or less, the attenuator design should be in accordance with TL-2 criteria. A project with a design speed of greater than 70 km/h will require an attenuator design which should be in accordance with TL-3 criteria. Attenuators shielding obstructions located between roadway facilities with different design speeds (e.g. in gore areas) should be in accordance with the Test Level requirement for the higher design speed.

The required attenuator width designation is based on the width of the obstruction. There are three standard widths available. They are as follows:

1. W1. This attenuator width is required for obstructions that are not more than 900 mm wide.

2. W2. This attenuator width is required for obstructions that are more than 900 mm wide but less than or equal to 1800 mm wide.

3. W3. This attenuator width is required for obstructions that are more than 1800 mm wide but less than or equal to 2400 mm wide.

The Type ED impact attenuator is limited to the W1 width only. Width requirements greater than W1 will necessitate the selection of a Type R1 or a Type R2 impact attenuator.

The Type LS impact attenuator is limited to the W1 width only, and shall be in accordance with TL-1 criteria. Width requirements greater than W1 will necessitate the selection of a Type R2 or a Type CR impact attenuator which should be in accordance with TL-2 criteria.

For all other impact attenuator types, if the obstruction width is greater than 2400 mm, the obstruction should be shielded by a special attenuator design, altered so the width is less than or equal to 2400 mm, or moved to a location where shielding is not required.

Figure 49-6C illustrates the space requirements for approved impact attenuators. On roadways with a shoulder section, the attenuator footprint shown on the figure should not encroach onto the usable shoulder, as defined in Chapters Fifty-three, Fifty-four, or Fifty-five, as appropriate. On roadways with curbs, the attenuator footprint should not encroach onto the 0.5 m appurtenance-free zone, as discussed in Section 49-2.0. If the roadway section includes a sidewalk, the attenuator footprint should not encroach upon the sidewalk to reduce the remaining sidewalk width to less than 1.2 m. If the attenuator footprint violates any of the above encroachment limits, the obstruction should be shielded with a roadside barrier, altered so the footprint encroachment is satisfactory, or moved to a location where shielding is not required.

49-7.0 PIER/FRAME BENT COLLISION WALLS

Collision walls should be provided on all new construction and reconstruction projects where the traffic face of the pier is not completely protected by guardrail or where there is a gap between adjacent piers that is not protected by guardrail.

49-7.01 Application

Where there is a frame bent (i.e., pier composed of columns) for an overhead structure, a collision wall should be constructed between the columns. For twin overhead structures, a collision wall should be constructed between the twin frame bents/piers.

49-7.02 Design

The following provides the Department's design criteria for collision walls:

1. Wall Height and Thickness. The minimum height of the collision wall above the shoulder/ground surface is 840 mm, and the minimum thickness of the collision wall is equal to the thickness of the adjacent piers or bents. The height of the collision wall should be increased to match the height of adjacent concrete median barrier walls.

2. Traffic Face Geometry. The traffic side face of the collision wall is a vertical shape.

3. Footing Design. The footing for the collision wall is 1200-mm wide by 300-mm thick with the bottom 900 mm below the ground line. A longitudinal keyway is required at the top of the footing. The width of the keyway is equal to 1/3 the thickness of the wall, a minimum of 200 mm, and with a depth of 75 mm.

4. Reinforcing Steel. For the wall reinforcing steel, the longitudinal reinforcing steel will be #13 bars @ 300-mm spacings; the vertical reinforcing steel will be #16 bars @ 300-mm spacings; and the horizontal reinforcing steel at the top of the wall will be #13 bars @ 300-mm spacings.

5. Impact Attenuators For Median Piers/Frame Bents. Impact attenuators are required at both ends of median piers/frame bents for single overhead structures. For twin overhead structures an impact attenuator is required at the incoming end of the "first" structure and the outgoing end of the "second" structure on multi-lane highways.

6. Existing Collision Walls. Existing collision walls which are less than 840 mm above the shoulder/ground in height should be extended to 840 mm by grouting vertical #16 reinforcing bars @ 300-mm spacings into the top of the existing wall along both faces and pouring concrete to the necessary height.

7. Typical Collision Wall Detail. Figure 49-7A illustrates typical details of a new collision wall.

49-8.0 BRIDGE RAIL ENDS

49-8.01 Curved W-beam Guardrail System

The curved W-beam guardrail system is composed of two subsystems. The first is the curved W-beam guardrail terminal system, which is used to terminate a guardrail run, where the run is interrupted by a driveway. The second subsystem is the curved W-beam guardrail connector system, which is used to connect guardrail located along a main roadway to guardrail or a guardrail end treatment located along an intersecting public road approach. Each subsystem contains different types which the designer can specify based upon site conditions.

The designer should note that the area behind the curved W-beam guardrail system should be cleared of all fixed objects which constitute a hazard as shown in the INDOT *Standard Drawings*.

49-8.02 Bridge Rail End Protection

AASHTO Specifications require that all bridge railing ends be protected from direct collision by traffic. The type and the amount of protection required is determined by the location of the bridge rail end relative to the clear zone and should be in accordance with Figure 49-8A, Bridge Rail End Protection Requirements.

The required length of bridge approach guardrail, including the guardrail transition, for both shoulders on 2-lane, 2-way highways and outside shoulders on multi-lane divided highways is based on the clear zone requirements for the roadway and the design speed. The calculated length is rounded up to the nearest whole multiple of 1.905 m. The lengths given in this Section are those required to protect the end of the bridge rail only and should be considered minimum requirements. All hazards adjacent to the bridge rail end should be considered where bridge approach guardrail length is computed.

49-8.03 Driveways and Public Road Approaches

The designer should make every effort to relocate or close driveway(s) that prohibit the installation of the required bridge approach guardrail and guardrail end treatment. Because this will not always be practical, each case will be determined on a case-by-case basis, with emphasis placed on providing the maximum protection practical consistent with the restrictions. Where such a driveway or public road approach cannot be relocated, the designer should specify the appropriate curved W-beam guardrail system, in accordance with the INDOT *Standard Drawings* and the guidelines contained herein. A minimum of 7.62 m of W-beam guardrail should be provided between the guardrail transition Type TGB and the curved W-beam guardrail system. Where this is not practical, the designer should specify a concrete bridge railing transition Type WGB and a guardrail transition Type WGB, instead of the Type TGB, to connect the concrete bridge rail to the curved W-beam guardrail system. The following Section discusses typical site condition requirements with the appropriate guardrail treatments for driveways and approaches. The appropriate guardrail layout at, and in advance of, the driveway and public road approach is dictated by the control line, which is established by the clear zone and the guardrail runout length (L_R).

49-8.03(01) Driveways

Except as noted below, a Type 1 or Type 4 curved W-beam guardrail terminal system should be used depending on the system radius required to meet the driveway radius. The designer needs to consider the following:

1. Type 5 Anchor (Located Beyond the Control Line). Where the Type 5 anchor of the curved W-beam guardrail terminal system, as shown in Figure 49-8B, Driveway Application (Beyond the Control Line), falls entirely beyond the control line, the bridge approach guardrail should be terminated at that point. However, the area in advance of the guardrail, bounded by the edge of travel lane and the control line, must be traversable. The designer must also show the additional grading on the plans.

2. Type 5 Anchor Located Partially or Entirely Within the Control Line. Where the Type 5 anchor of the curved W-beam guardrail terminal system, as shown in Figure 49-8C, Driveway Application (Inside the Control Line), falls partially or entirely within the control line, the guardrail run should be continued on the other side of the driveway to the point of need. This will require another curved W-beam guardrail terminal system along the other side of the driveway, additional W-beam guardrail along the roadway shoulder in advance of the driveway and an appropriate guardrail end treatment. This advance guardrail should be extended from the end of the curved W-beam guardrail terminal to the point of need and then connected to the guardrail end treatment. However, if this guardrail length required in advance of the driveway is less than 30 m, then the guardrail run and curved W-beam guardrail terminal system in advance of the drive will not be required. However, the area in advance of the guardrail, bounded by the edge of the travel lane and the control line, must be traversable. The designer must show this additional grading on the plans.

3. Restricted Right of Way. When the obtainable right of way is insufficient to use the normal configuration, a modified version of the curved W-beam guardrail terminal system should be used. These modified versions have shorter "legs" along the side of the driveway and are designated as Types 2, 3, 5 and 6, as shown in the INDOT *Standard Drawings*. Types 2 and 5 are 1.905 m (one panel) shorter than the "standard" version, and Types 3 and 6 are 3.810 m (two panels) shorter than the "standard" version. The designer should choose the appropriate type based on the system radius required to meet the driveway radius and the amount of shortening required by the restricted right of way. The restrictions concerning the location of the Type 5 anchor and the need for additional guardrail in advance of the driveway are still applicable to this situation.

Examples of restricted right of way include avoidance of a wetland or other environmentally sensitive area or a lawn. An example of an area where additional right of way should probably be purchased to avoid removing panels is agricultural land. On 3R projects, the designer is reminded to consider the criteria in Section 55-5.04(02) Item 5. It may be possible to shorten the guardrail run or eliminate the guardrail terminal system.

49-8.03(02) Public Road Approaches

A Type 1 or Type 2 curved W-beam guardrail connector should be used depending on the system radius required to meet the approach radius. The designer should consider the following:

1. Curved W-Beam Guardrail Connector System (End Located Beyond the Control Line). Where the end of the curved W-beam guardrail connector system falls beyond the control line, as shown in Figure 49-8D., Public Road Approach Application (Beyond the Control Line), no additional guardrail is required along the public road approach. An appropriate guardrail end treatment should be used to attach to the end of the curved W-beam guardrail connector system. In addition, the area in advance of the guardrail, bounded by the edge of travel lane and the control line, must be traversable. The designer must also show the additional grading on the plans.

2. Curved W-Beam Guardrail Connector System (End Located Inside the Control Line). Where the end of the curved W-beam guardrail connector system falls within the control line, as shown in Figure 49-8E, Public Road Approach Application (Inside the Control Line), additional guardrail will be required from the end of the curved W-beam guardrail connector system to the control line, terminated by an appropriate guardrail end treatment.

3. Guardrail Requirements for Public Road Approach. If additional guardrail is needed to meet the clear zone requirements along the public road approach, then this guardrail should extend from the end of the curved W-beam guardrail connector system to the point of need along the public road approach and be terminated with an appropriate guardrail end treatment.

49-8.04 Unfavorable Site Conditions

The designer will frequently encounter site conditions which prohibit or restrict the use of these treatments. The designer should make every effort to obtain the necessary driveway/approach relocation, additional rights of way and clearance for fixed obstacles to provide the suitable protection. If these efforts are not practical, then a special design may be necessary, and the designer should contact the Contracts and Construction Division's Standards Section for assistance.

49-8.05 Median Shoulder Bridge Approach Guardrail Lengths

Lengths of median shoulder bridge approach guardrail are based on the clear zone requirements for the roadway. The entire length of the median shoulder bridge approach guardrail, exclusive of the TGB transition, is double faced. The required minimum lengths are shown in Figure 49-8F, Median Bridge Approach Criteria. The flare and offset given is the desired layout of the guardrail. The length of bridge approach guardrail should be recomputed for site conditions other than those assumed and listed in Figure 49-8F.

49-9.0 TRUCK HEIGHT 1145-mm CONCRETE BARRIER

Section 49-4.05 describes the warrants for concrete barriers in medians. This section describes the warrants for 1145-mm height concrete barriers for all applications. The 1145-mm height barrier may be warranted where there is a high volume of truck traffic, above deep water, on high-occupancy land use areas, on high fills, across deep ravines or for a combination of these factors. The procedure for determining whether or not the 1145-mm barrier is warranted is described below.

49-9.01 Performance Level Selection

The basic parameter for concrete barrier selection is the performance level required at the site. This is a function of the following:

1. highway design speed;
2. traffic volumes (especially truck volumes);
3. barrier offset;
4. highway geometry (grades, horizontal curvature);
5. height of bridge deck; and
6. type of land use below bridge deck.

This Section presents the detailed methodology for determining the performance level selection for concrete barriers. The methodology has been adapted from the AASHTO publication *Guide Specifications for Bridge Railings*, 1989. The *Guide Specifications* methodology is based on a benefit/cost analysis which considers occupant safety, vehicular types, highway conditions and costs. The overall objective is to match the concrete barrier performance level (and therefore costs) to site needs. Because of the similarities between the potential safety hazards from penetrating a bridge railing or a median/shoulder barrier, INDOT also applies this methodology to the performance level selection for median/shoulder barriers.

The methodology presented herein references PL-2 and PL-3 concrete barriers of 840 mm height.

This is consistent with the terminology used in the *Guide Specifications*. For INDOT application of this Section, the following will apply:

1. A PL-2 barrier refers to an 840-mm height concrete barrier.
2. A PL-3 barrier refers to an 1145-mm height concrete barrier.

Section 61-6.0 presents a more detailed discussion on the performance level selection for bridge railings.

Figure 49-9A, 49-9A, Threshold Warrants for PL-2/PL-3 Concrete Barriers (Divided Highways or Undivided Highways with ≥ 5 Lanes), Figure 49-9B, Threshold Warrants for PL-2/PL-3 Bridge Railings (Undivided Highways with ≤ 4 Lanes), and Figure 49-9C, Threshold Warrants for PL-2/PL-3 Concrete Barriers (Ramps or One-Way Highways), apply directly to highways on tangent, on level roadways, with deck surfaces approximately 10.5 m above the under structure ground or water surface, and with low-occupancy land use or shallow water under the structure. The truck volume used to determine the performance level should be the design year average daily truck traffic.

For highway conditions that differ from those above, the truck volume should be adjusted by the correction factors in Figure 49-9D, Grade Traffic Adjustment Factor (K_g) and Curvature Traffic Adjustment (Factor (K_c), and Figure 49-9E, Traffic Adjustment Factor (K_s) (Deck Height and Under-Structure Shoulder Height Conditions). These correction factors are for highway grade (K_g), horizontal curvature (K_c), and deck/shoulder height and under-structure conditions (K_s). The high-occupancy land use referred to in Figure 49-9E applies to sites where there is a relatively high probability for injury or for loss of human life. The low-occupancy land use applies to sites where the probability for injury or loss of human life is relatively low.

49-9.01(01) Bridge Railings / Shoulder Barriers

Where a PL-2 or PL-3 bridge railing / shoulder barrier will be used, the following procedure will apply to the selection of the appropriate bridge railing.

1. Determine adjustment factors K_g and K_c from Figure 49-9D, Grade Traffic Adjustment Factor (K_g) and Curvature Traffic Adjustment (Factor (K_c), and K_s from Figure 49-9E.

2. Calculate the adjusted average daily truck volume by multiplying the design year average daily truck volume (total for both directions) by the three adjustment factors shown below.

   Adjusted average daily truck volume = (design year avg. daily truck volume) (K_g) (K_c) (K_s).

3. Locate the appropriate table for the type of roadway on which the bridge is located.

4. Locate the line in the table that describes the site conditions (percent trucks and bridge railing offset).

5. Move across to the column for the highway design speed.

6. Locate the adjusted average daily truck volume value on the table.

7. If the calculated adjusted average daily truck volume value (Step 2) exceeds the adjusted average daily truck volume value from the table (Step 6), then a PL-3 bridge railing is warranted. If it is less, then a PL-2 bridge railing is warranted.

Each side of the bridge should be checked against these criteria. This is especially important for those bridges on horizontal curves. If the PL-3 bridge railing is warranted on one side of the bridge, it should also be used on the other side.

See Section 49-9.03 for example calculations on the selection of a PL-2 or PL-3 bridge railing.

On minor bridge rehabilitation projects which do not include bridge deck replacement or deck widening and currently have a crashworthy PL-2 bridge railing, the existing bridge railing need not be upgraded to a PL-3 bridge railing even though the warrants for the PL-3 bridge railing are satisfied. If there is no significant history of truck accidents, the installation of the PL-3 bridge railing should be deferred until the time of deck replacement or deck widening. However, if truck accidents are a problem, consideration should be given to installing the PL-3 railing on the rehabilitation project along with countermeasures to reduce the truck accident problem.

49-9.01(02) Median Barriers

If a median barrier is warranted in a freeway based on the criteria shown in Figure 49-9H, 1145-mm Concrete Median Barrier (Example 49-9.3), an 1145-mm height concrete barrier should be used.

The following procedure should be used to determine if an 1145-mm height concrete median barrier is warranted in an expressway.

1. Determine adjustment factors K_g and K_c from Figure 49-9D, Grade Traffic Adjustment Factor (K_g) and Curvature Traffic Adjustment (Factor (K_c). Use $K_s = 0.7$.

2. Calculate the adjusted average daily truck volume by multiplying the design year average daily truck volume (total for both directions) by both adjustment factors:

Adjusted avg. daily truck volume = (design year avg. daily truck volume) (K_g) (K_c) (0.7).

3. Locate the line in Figure 49-9A, Threshold Warrants for PL-2/PL-3 Concrete Barriers (Divided Highways or Undivided Highways with $\geq$ 5 Lanes), that describes the site conditions (percent trucks and median barrier offset).

4. Move across to the column describing the highway design speed.

5. Locate the adjusted average daily truck volume value on the table.

6. If the calculated adjusted average daily truck volume value (Step 2) exceeds the adjusted average daily truck volume value from the table (Step 5), then a PL-3 median barrier is warranted. If it is less, then a PL-3 median barrier is not warranted.

7. If a PL-3 median barrier is warranted, it should be used between logical termini, such as two bridge piers.

See Section 49-9.03 for an example calculation on the selection of a PL-2 or PL-3 median barrier.

It will be necessary to check the impact of the median barrier on intersection sight distance.

49-9.02 Barrier Design and Layout

The design and layout of the 1145-mm height barrier must be compatible with the geometric design of the highway and with the roadside safety criteria. These are discussed below.

49-9.02(01) Approach Barrier Type and Length of Need

Standard guardrail and guardrail-to-bridge railing transitions are normally used on the approach to a bridge. However, an 1145-mm concrete barrier is used when warranted, to contain large trucks which could depart from the roadway, resulting in a high risk of loss of life or severe injury to pedestrians or people in vehicles on crossroads or parallel roads. The 1145-mm concrete barrier should be used when all of the following conditions exist.

1. The warrants for the 1145-mm bridge railing have been satisfied. [See Section 49-9.01(01)].
2. The mainline or ramp has a radius of 437 m or less.

3. The design year ADT of the crossroad or parallel roadway below, which is within 40 m of the edge of the overhead travel lane, is equal or greater than 7,500 vpd.

4. The physical characteristics of the roadside are such that an errant truck that crashes through a standard bridge approach guardrail or shoulder barrier can reasonably be expected to reach the crossroad, parallel roadway, or other high-occupancy land use area below.

On existing facilities the accident data for the most recent 3 years should be obtained and analyzed. If an adverse truck accident history is found, consideration should be given to installing the 1145-mm barrier even if the listed warrants are not satisfied.

Consideration should also be given to installing the 1145-mm concrete barrier on the bridge approach on new facilities in locations where driver expectations are violated such as where a steep down grade or long tangent section in advance of a curve over a crossroad will be constructed.

The length of need for a 1145-mm barrier or standard guardrail before and beyond the bridge is determined from the length-of-need equations for roadside barriers (see Section 49-5.0). The length of the 1145-mm barrier should be based on the barrier length of need or the tangent runout path, whichever is less. Where a roadside barrier is warranted beyond the 1145-mm concrete barrier, the additional barrier will normally be standard W-Beam guardrail. Where the 1145-mm approach barrier is used, it must be tapered down to a height of 840 mm. Any additional standard guardrail beyond the concrete barrier must include a proper guardrail transition.

The minimum length of need for an 1145-mm concrete barrier in the median can be determined as discussed above. Other logical points of termination that should be considered include bridge piers or parapets, median crossovers or, in some cases, the entire project length.

49-9.02(02) End Treatments

End treatments for the 1145-mm height barrier are as follows:

1. Bridge Railing. Unless transitioned to a roadside barrier, the end of the 1145-mm bridge railing will be shielded by an appropriate impact attenuator. This applies whether the end is inside or outside of the clear zone.

2. Concrete Barrier. The 1145-mm concrete barrier should be tapered down to the common concrete barrier height (840 mm) where it is connected to the concrete barrier as shown in the INDOT *Standard Drawings*. This taper should be accomplished outside the area where the 1145-mm barrier is warranted. If it does not connect to the common-height concrete barrier, the ends must be tapered down to a height of 840 mm and terminated with an appropriate impact attenuator.

3. Transition From 1145 mm to 840 mm. The transition from the truck-height section to the

common-height section should be sloped 30:1 or flatter.

49-9.02(03) Horizontal Sight Distance

The use of an 840-mm or 1145-mm height barrier may limit stopping sight distance (SSD) on the inside of horizontal curves. Therefore, the designer should check the SSD on horizontal curves and determine if the required SSD is available (see Section 43-4.0). If SSD requirements are not met, the designer should evaluate the impacts of the reduced SSD on safety and, if appropriate, seek a Level One design exception (see Section 40-8.0). If, for example, safety is significantly reduced, then the 1145-mm barrier may not be appropriate.

49-9.02(04) Interchange Entrance Ramps

Drivers entering a freeway need sufficient sight distance to locate gaps in the traffic stream in which to merge. Depending upon many factors at the site, the presence of an 1145-mm barrier could interfere with the sight distance of an entering driver. Therefore, the designer should check entrance ramps to ensure that adequate sight distance is available for the merge maneuver.

49-9.02(05) Median Barriers with C-D Roads

A concrete barrier may be warranted between a highway mainline and a collector-distributor road. In this case, the 1145-mm concrete barrier should not be used because of the importance of sight distance.

49-9.03 <u>Example Calculations for PL-2/PL-3 Barriers</u>

Section 49-9.01 presents the methodology used to determine whether a PL-2 or PL-3 barrier is appropriate for the site conditions. This Section presents three example problems to illustrate the use of the methodology.

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<u>Example 49-9.1</u> Figure 49-9F illustrates the example.

Given: Twin structures on an urban freeway.
Design speed = 100 km/h
Design year average daily truck volume = 7,000 vpd
Percent trucks = 20%
Bridge railing offset: Median shoulder = 1.2 m

Outside shoulder = 3.3 m

Horizontal curve radius = 580 m

Grade = -3% northbound

High-occupancy land use

Deck height above under structure surface = 7.3 m

Problem: Determine whether a PL-2 or PL-3 bridge railing is appropriate at the site.

Solution: Northbound outside bridge railing:

From Figure 49-9D, Grade Traffic Adjustment Factor (K_g) and Curvature Traffic Adjustment (Factor (K_c), $K_g = 1.25$ and $K_c = 1.0$

From Figure 49-9E, Traffic Adjustment Factor (K_s) (Deck Height and Under-Structure Shoulder Height Conditions), $K_s = 0.9$

Adjusted design year average daily truck volume = (7,000)(1.25)(1.0)(0.9) = 7,875 vpd.

From Figure 49-9A, Threshold Warrants for PL-2/PL-3 Concrete Barriers (Divided Highways or Undivided Highways with ≥ 5 Lanes), the adjusted average daily truck volume threshold is 6,600 vpd.

7,875 > 6,600; therefore, a PL-3 bridge railing is warranted.

Northbound median bridge railing:

From Figure 49-9D, $K_g = 1.25$ and $K_c = 1.0$

From Figure 49-9E, $K_s = 0.9$

Adjusted design year average daily truck volume = (7,000)(1.25)(1.0)(0.9) = 7,875 vpd.

From Figure 49-9A, the adjusted average daily truck volume threshold is 6,300.

7,875 > 6,300; therefore, a PL-3 bridge railing is warranted.

Southbound outside bridge railing:

From Figure 49-9D, $K_g = 1.0$ and $K_c = 1.0$

From Figure 49-9E, $K_s = 0.9$

Adjusted design year average daily truck volume = (7,000)(1.0)(1.0)(0.9) = 6,300 vpd.

From Figure 49-9A, the adjusted average daily truck volume threshold is 6,600 vpd.

6,300 < 6,600; therefore, a PL-2 bridge railing is warranted.

Southbound median bridge railing:

From Figure 49-9D, $K_g = 1.0$ and $K_c = 1.0$
From Figure 49-9E, $K_s = 0.9$

Adjusted design year average daily truck volume = (7,000)(1.0)(1.0)(0.9) = 6,300 vpd.

From Figure 49-9A, the adjusted average daily truck volume threshold is 6300 vpd.

6,300 = 6,300; therefore, a PL-2 bridge rail is warranted.

Example 49-9.2 Figure 49-9G illustrates the example.

Given: Structure on a rural 2-lane arterial.
Design speed = 100 km/h
Design year average daily truck volume = 3,165 vpd
Percent trucks = 15%
Bridge rail offset for adjacent traffic = 3.3 m
Bridge rail offset for opposing traffic = 3.3 m + 3.6 m = 6.9 m
Horizontal curve radius = 435 m
Grade = 3% eastbound
Structure over shallow water
Deck height above under structure surface = 7.3 m

Problem: Determine whether a PL-2 or PL-3 bridge railing is appropriate.

Solution: Westbound right-side bridge railing:

From Figure 49-9D, Grade Traffic Adjustment Factor (K_g) and Curvature Traffic Adjustment (Factor (K_c), $K_g = 1.25$ and $K_c = 1.33$
From Figure 49-9E, Traffic Adjustment Factor (K_s) (Deck Height and Under-Structure Shoulder Height Conditions), $K_s = 0.7$

Adjusted design year average daily truck volume = (3,165)(1.25)(1.33)(0.7) = 3,683 vpd.

From Figure 49-9B, Threshold Warrants for PL-2/PL-3 Bridge Railings (Undivided Highways with $\leq$ 4 Lanes), the adjusted average daily truck volume threshold is 4,200 vpd.

3,683 < 4,200; therefore, a PL-2 bridge railing is warranted for the outside westbound bridge railing.

Solution: <u>Eastbound right-side bridge railing</u>:

From Figure 49-9D, $K_g = 1.0$ and $K_c = 2.0$
From Figure 49-9E, $K_s = 0.7$

Adjusted design year average daily truck volume = (3,165)(1.0)(2.0)(0.7) = 4,431 vpd.

From Figure 49-9B, the adjusted average daily truck volume threshold is 4,200 vpd.

4,431 > 4,200; therefore, a PL-3 bridge railing is warranted.

Because it is warranted on one side of the bridge, a PL-3 bridge railing should also be used on the other side.

<u>Example 49-9.3</u> See Figure 49-9H, 1145-mm Concrete Median Barrier (Example 49-9.3).

Given:
6-lane urban freeway
Design speed = 110 km/h
Design year average daily truck volume = 8,000 vpd
Percent trucks = 10%
Median width = 7.3 m
Median barrier offset = 3.3 m
Horizontal curvature = tangent
Grade = 3% eastbound

Problem: Determine whether a PL-2 or PL-3 concrete median barrier is appropriate.

Solution: <u>Eastbound traffic</u>:

From Figure 49-9D, Grade Traffic Adjustment Factor (K_g) and Curvature Traffic Adjustment (Factor (K_c), $K_g = 1.0$ and $K_c = 1.0$. $K_s = 0.7$.

Adjusted design year average daily truck volume = (8,000) (0.7) (1.0) (1.0) vpd
= 5,600 VPD

From Figure 49-9A, Threshold Warrants for PL-2/PL-3 Concrete Barriers (Divided Highways or Undivided Highways with ≥ 5 Lanes), the adjusted average daily truck volume threshold is 6,400 vpd.

5600 < 6400; therefore, a PL-3 median barrier is not required.

<u>Westbound traffic</u>:

From Figure 49-9D, $K_g = 1.25$ and $K_c = 1.0$. $K_s = 0.7$.

Adjusted design year average daily truck volume = (8,000) (0.7) (1.25) (1.0) = 7,000 vpd.

From Figure 49-9A, the adjusted average daily truck volume threshold is 6,400 vpd.

7,000 > 6,400; therefore, a PL-3 median barrier is required.

49-10.0 GUIDE TO THE ROADSIDE COMPUTER PROGRAM

This Section supplements the information in Appendix A of the 1988 AASHTO *Roadside Design Guide* and in the “README” file of the ROADSIDE computer program. It provides more detailed information and guidance on the use of ROADSIDE and an expanded listing of recommended severity indices and an example of a sensitivity analysis.

It should be noted that this program was written using the English units. At the time of publication of the *Indiana Design Manual*, the program had not yet been converted to metric units; therefore, this Section has been prepared using English units. For the purpose of this Section, the designer should use the following conversion factors:

1. <u>Speed</u>. 1 km/h = 0.6215 mph

2. <u>Length</u>. 1 meter = 3.2808 ft

3. <u>Horizontal Curves</u>. Radius (meters) = 1746.8/D, where D is the degree of curvature (100-ft arc definition).

49-10.01 Introduction

The program, ROADSIDE, is a useful tool for highway engineers making decisions for the design of roadsides and the placement of highway hardware. It aids the designer in selecting an alternative treatment which offers the greatest anticipated return for safety benefits for funds expended. ROADSIDE is the microcomputer version of the Cost-Effectiveness Selection Procedure (Appendix A) in the 1988 AASHTO *Roadside Design Guide*. The program is written in Quick Basic 4 and is not copyrighted. Thus, modifications to the program can be made if the user has an understanding of basic programming and the assembled language of the program.

49-10.01(01) Using ROADSIDE

With the PC turned on, insert the ROADSIDE diskette into the microcomputer. At the DOS prompt, change to the appropriate drive, type ROADSIDE and press Enter.

The program then reads the data files containing the lateral extent of encroachment probabilities and displays a note on the screen to that effect.

The Basic Input Data screen (Figure 49-10A) and global values are then shown, with an inquiry to the user regarding the value to be used. If no changes to the basic input data are desired, type N (no) and press Enter. The severity index versus cost relationship is displayed next for the user's information. Press Enter to continue.

The Variable Input Data screen (Figure 49-10B) is the last screen displayed. All data entry occurs on this screen. To enter data, type the appropriate line number from the left-hand margin and press Enter. A new screen will then be displayed showing the current value and asking the user to enter the new value for the field in question. All calculations are automatically made as the user inputs values for each variable. Whenever an input variable is changed, all calculations using that variable are automatically made and the new results are displayed.

The Command Menu at the bottom of the Variable Input Data screen identifies the function keys listed below that are used in ROADSIDE.

49-10.01(02) Function Keys

The following function keys are used in the program:

1. Function Key 1. This key will print a copy of the Variable Input Data screen and the resultant computations. The printout contains some information that does not appear on the computer screen. The computer screen was modified so all data entry can be made on a

single screen.

2. Function Key 2. This key will store the problem variables and basic input data.

3. Function Key 3. This key will retrieve a previously stored problem. The user will be given two or three options. If the problem was stored with the original default values, the user may have the problem recalled to the screen using the default data or using the basic input data values from the last problem shown on the screen (called the "current" values). If the problem was stored using altered values, then it may be recalled using those values ("dataset" values), using the "default" values, or using the basic input values that were used on the last problem shown on the screen ("current" values).

4. Function Key 4. This key will let the user access the HELP menu which contains detailed information on every aspect of ROADSIDE.

5. Function Key 5. This key will display, and allow the user to change, the basic input (global) values.

6. Function Key 6. This key will display the relationship between severity index and cost as derived from the accident costs included in the basic input values.

7. Function Key 7. This key will list all file names on the ROADSIDE disk.

8. Function Key 8. This key lists the percentage of accident types included for each severity index value.

9. Function Key 9. This key will, for computers with graphic display capability only, provide a sketch of the highway roadside, and hazard parameters. The "Print Screen" key will allow the user to obtain a hard copy of this sketch if a dot matrix printer is used. A "daisy wheel" will not print correctly.

10. Function Key 10. This key is used to exit the program. No data are stored via this function. Data should be stored using Function Key 2.

49-10.02 Basic Input Data

The first input screen (Figure 49-10A) shows all default values. While these numbers represent the best judgement of the program developers, the user of this program has the option to change any default value as deemed appropriate based on new data or on local conditions. If no changes are made in these variables, the program then prints out accident costs for each severity index based on the default accident costs by accident type.

The swath width is the effective width of an encroaching vehicle that is not tracking. Although this width naturally varies depending on vehicular length, width and yaw angle, a width of 12 feet is the default value used to represent a typical vehicle. The yaw angle, shown in Figure 49-10C, is defined as the angle between the direction the vehicle is traveling and the direction the vehicle is pointing. This value may be changed if desired, but it is considered both reasonable and representative for analysis purposes.

Accident costs are assigned to each of three categories of accidents — fatal, injury and property damage only (PDO). Injury and PDO accidents are further divided into different levels of severity. The default values in the program may be changed, but it is recommended that the default values be used for lack of more current information. Accident costs used in economic evaluations differ significantly between agencies. The default values in the model were selected as median values. Should they be changed, the values assigned to these, especially fatal accidents, will have a significant effect on the numerical values and the calculated cost-benefit ratios, but it will usually not change the relative ranking of the alternatives being considered. The effect of using one set of values over another can be assessed using a sensitivity analysis. This procedure is illustrated with the example problem where the same alternatives are analyzed using the default accident costs included in Figure 49-10A and with the FHWA-recommended costs from FHWA Technical Advisory T 7570.1.

49-10.03 Variable Input Data

The second input screen (Figure 49-10B) in the program includes specific roadway and roadside characteristics that must be entered by the user. The program contains Lateral Extent of Encroachment Probability tables for 40, 50, 60 and 70 mph, and adjustment coefficients for horizontal curvature and grade.

The following subsections describe each of the input data and explain how they are used in this program. Figure 49-10D is provided for quick reference.

49-10.03(01) Title

Each alternative or each iteration should be assigned a unique title if it will be saved for later retrieval and comparison to other alternatives. When saving an alternative, a unique file name will also be required. The title and file name need not be the same.

49-10.03(02) Traffic Volume and Growth

Line	Input Data	Units
2	*Traffic Volume*	*two-way ADT*
	Growth Rate	*percent*

Enter the current daily 2-way traffic volume and an estimated annual growth rate. The traffic growth rate is entered as a percentage (0 to 10%). In the absence of other guidance, a traffic growth rate of 2.0% is suggested.

The model assumes the characteristics of the highway facility are uninterrupted flow with no interaction among vehicles in the traffic stream. When traffic volumes reach capacity, the characteristics change to interrupted flow and the volume-encroachment relationship is no longer valid. Therefore, a default value limits maximum traffic volume to 10,000 vehicles per lane per day. Volumes higher than 10,000 are reduced to 10,000 vehicles per lane per day in the first year only. The program does not limit or omit volumes which may exceed 10,000 vehicles per lane per day during the remaining project life. ROADSIDE does not assign traffic to individual lanes on multi-lane highways. This is discussed in Section 49-10.03(03).

Multi-lane facilities in many cases will operate at uninterrupted flow except for peak hours. The 10,000 limit may be too low because the facility will operate at uninterrupted flow the majority of the time. A higher limit of 15,000 vehicles per lane per day may be used for multi-lane highways.

Traffic volume is a significant factor for determining user costs; therefore, using accurate volumes is important. The growth rate usually does not significantly affect the user and agency costs. A general rate readily available should be used because of this.

49-10.03(03) Roadway Type

Line	Input Data	Units
3	*Roadway Type*	*undivided (U), divided (D), one-way (O)*
	Lanes of Adjacent Traffic	*number of lanes*
	Width of Each Lane	*feet*

Enter the type of highway being analyzed. Three options exist — divided, undivided and one-way. For undivided highways, encroachments on one side of the road by both adjacent and opposing traffic are calculated. Encroachments from the opposite direction are not computed on divided and one-way highways. The number of lanes of adjacent traffic and the width of each lane must also be entered. Adjacent traffic is defined as all lanes traveling in the same direction on the roadway next to the obstacle. A 2-lane undivided highway will have one adjacent lane of traffic whereas a 4-lane divided highway will have two adjacent lanes.

The obstacle can be located in the median or to the right of the traveled way. The model does not recognize whether the encroachments occur on the inside (median) or outside of the roadway. The user should treat the median as if it is a roadside. An analysis in the median may also require separate program runs so that encroachments are considered from both directions.

The total traffic volume is split equally between both directions of travel, except for one-way roadways or ramps. The directional volume is assigned to the lane closest to the obstacle. In actuality, there is a distribution of total traffic between the travel and passing lanes for a multi-lane highway. Most of the traffic in the travel lane will be an additional 12 feet from a hazard located in the median. Therefore, the number of encroachments may be overestimated for a median-side analysis, where the lane closest to the obstacle normally carries lighter traffic volume. An analysis more representative of the actual lane distribution could be obtained by running the program separately for each lane. Figure 49-10E can be used to select approximate lane distributions for 4- and 6-lane highways. With each program run, the only input variables that would change are traffic volume and the distance to the obstacle. An alternative method is to apply the appropriate factor in Figure 49-10F and Figure 49-10G; this provides the same answer as the sum of separate program runs.

49-10.03(04) Geometric Adjustment Factors

Line	Input Data	Units
4	*Roadway Curvature Adjustment*	*degrees*
	Roadway Grade Adjustment	*percent*

There are two geometric adjustment factors for the encroachment rate. These are listed below:

1. Roadway Curvature Factor. Curves to the right (for adjacent traffic) are assigned a (+) sign and can increase the basic encroachment rate by a factor of 2 (maximum) for curves of 6 degrees or sharper. Curves 3 degrees or flatter do not increase the basic rate.

   Curves to the left (for adjacent traffic) are assigned a (-) sign and can increase the basic encroachment rate by a factor of 4 (maximum) for curves of 6 degrees and sharper. Curves of 3 degrees and flatter do not change the basic rate. ROADSIDE selects the appropriate factor when the degree of curvature is entered.

2. Roadway Grade Factor. Negative grades (downgrades) in the direction of adjacent traffic increase the basic encroachment rate by a factor of 2 for 6% or steeper grades. Downgrades of 2% or less do not affect the basic rate. The appropriate factor is selected when the grade is entered by the program user.

   For example, a tangent highway section 1/3 mile in length with 6,000 ADT will have a calculated value of 1 encroachment for two years (1/3 mile x 3,000 ADT per direction x

0.0005 encroachment rate x 2 years = 1). This is neglecting opposite direction encroachments. If that highway section was on a 6-degree curve with a 6% grade, there would be 8 encroachments on the outside downhill curve [4 (curve factor) x 2 (grade factor) x 1 encroachment = 8] and 2 encroachments on the inside uphill curve [1 (curve factor) x 2 (grade factor) x 1 encroachment = 2].

49-10.03(05) Encroachment Rate

Using the data up to this point (lines 2, 3 and 4), the program automatically computes the total number of encroachments. An encroachment begins when a vehicle leaves the roadway (i.e., crosses the edge of the travel lane and/or moves onto the shoulder). The number of encroachments is shown for the total adjacent and opposing traffic (see Figure 49-10B). Adjustments are made for roadway characteristics (horizontal and vertical alignment) which will increase the number of encroachments.

The user adjustment factor allows the user to modify the basic rate if there are site specific conditions or an accident history that warrant a change. The user factor can be used to adjust the predicted number of encroachments with actual conditions or historical data.

As mentioned earlier, the user factor could be used to adjust for encroachments on multi-lane highways. This saves a step in running the program once versus several times for each lane. Figures 49-10F and 49-10G provide factors to use for analyzing either the median or outside of either a 4- or 6-lane highway.

49-10.03(06) Design Speed

Line	Input Data	Units
6	*Design Speed*	*miles per hour*

The design speed of the roadway is used to select a lateral-extent-of-encroachment probability curve. Curves for speeds of 40, 50, 60 and 70 mph are used in the program. For any input speed less than 40 mph, the 40-mph curve is used; the 50-mph curve is used for speeds between 40 and 50; the 60-mph curve is used for speeds between 50 and 60, and the 70-mph curve is used for speeds above 60 mph. These curves assume flat side slopes and underestimate the lateral extent of encroachment when slopes steeper than 10:1 exist. They may also overestimate the lateral distance a vehicle is likely to travel on a backslope. A design speed lower than the posted speed limit should not be used. At site specific locations, generally use speeds that closely approximate the actual or anticipated operating speed of the facility. At certain sites, such as some suburban highway sections with large peak hour volumes, the average operating speed may not accurately represent the design speed. In these cases, use the low-volume operating or running speed which represents the most likely condition for a single vehicle off roadway accident.

49-10.03(07) Hazard Definition

Line	Input Data	Units
7	*Hazard Offset from Driving Lane*	*"A" feet*
	Hazard Length (parallel to road)	*"L" feet*
	Hazard Width (perpendicular to road)	*"W" feet*

ROADSIDE defines a roadside hazard as a rectangle that is laterally offset from the edge of the driving lane a distance of A feet, is L feet long in the direction of travel, and W feet wide. The hazard can be a bridge pier, a large box culvert inlet and channel, an embankment, or a traffic barrier designed to shield a roadside obstacle or non-traversable terrain feature.

Defining the area of concern for multiple obstacles can be difficult. The program should not be run several times for each obstacle and composite costs added. Such an analysis implies a degree of accuracy the model lacks. In some cases the hazard may be behind another hazard (i.e., trees behind traversable ditch, 3:1 slope with trees at bottom, etc). In some cases there may be multiple hazards (trees on slope, culvert outlet on slope, etc). In defining these hazards, a single program run is accurate enough. This will require the user to select a rectangle that includes all significant hazards, a procedure similar to defining an area of concern for barrier layout (page 5-32, 1988 AASHTO *Roadside Design Guide*). For varying or multiple offset distances, an average offset distance should be used. The severity index may also need to be adjusted to account for various combinations of hazards (see severity index Section 49-10.03(09)).

User costs are sensitive to the offset distance and length of obstacle. The closer to the roadway and the longer the obstacle, the more chances for collisions. Agency costs are also sensitive to obstacle length. The width of the obstacle does not significantly influence costs.

49-10.03(08) Collision Frequency

Using the data supplied up to this point (lines 2 through 7), the program calculates the collision frequency. Once you have defined an object and determined how far it is from the ETL, the number of vehicles which hit the object are automatically calculated. The expected number of collisions with the hazard each year is the summation of collisions into the side, corner and longitudinal face of the hazard by adjacent and (where applicable) opposite-direction traffic. The input screen shows the initial collision frequency (impacts per year) for the whole object and for each location on the hazard impacted (face, side and corner). The collision frequency over the life of the project is only shown on the output screen.

Collision frequency is basically an accident rate for the object's exposure, because the number of impacts are determined over the length of the object. For example, a 1,000-ft length of guardrail, 8 ft from the ETL on a 6,000 ADT 2-lane roadway, will have an estimated number of 0.22926 impacts for the first year. Over five years, this equates into 1 accident (0.22926 x 5 years) for that 1000-ft section of guardrail.

49-10.03(09) Severity Index

Line	Input Data
9	*Severity Index for:* *upstream side of hazard (SU)* *downstream side of hazard (SD)* *upstream corner of hazard (CU)* *downstream corner of hazard (CD)* *longitudinal face of hazard (FACE)*

To convert accidents to costs, a severity index (SI) must be assigned to impacts with the hazard. Essentially, assigning a SI to an object is determining the relative cost per accident. The relationship between severity index and the percent accident type is shown on page A-12 of the *RDG*. For example, assigning a SI of 5.0 for a tree is predicting that resulting impacts will be 8% fatalities, 77% injuries, 15% PDO. Taking each percentage by accident costs (e.g., 8% x $500,000, etc.), the predicted cost per accident is $56,535.

ROADSIDE has no capability to select an appropriate SI and is dependent upon the user for this information. The more severe an object (higher SI), the higher the associated accident costs are. Once a SI is assigned to an object, the program automatically computes the resultant accident costs.

Impacts into a given object may have different outcomes based on where the vehicle hits. Therefore, adjustments can be made for impacts into the side of the hazard, the upstream and downstream (for 2-way traffic) corners of the hazard, and the face of the hazard. These will be equal for point objects such as trees and utility poles. For barriers, the severity of the accident will be less for a face impact than for a side or corner hit.

Figures 49-10H through 49-10P have been developed to provide more information to the user. Accident data was not used to develop the table. To determine SI's from accident records would require detailed accident data for each roadside object or obstacle. Unfortunately, accident reports seldom contain all the information needed to identify the object or obstacle struck in detail. The SI is a relative value, rather than an absolute or discrete number. It does not represent an impact into a specific object at the selected design speed, but rather an average estimated impact speed, given the selected design speed. This means that for most features there will be many low severity accidents included; vehicles that are nearly stopped before reaching the feature or striking it in such a way that occupants are not seriously injured. That is why the numbers are generally lower than the values in the 1977 *Barrier Guide*, which represented the severity of crashes at 60 mph. The tables were developed by ranking each common object by speed (e.g., different types of guardrail, etc).

The severity indices shown on Figures 49-10H through 10P incorporate ranges for each obstacle. The range covers other performance factors beyond those considered in the model. The user should read the information when selecting a value within the range. The ranking was based on the anticipated performance and intuitive judgement from engineers with backgrounds in safety, design and research. Based on historical data of relative relationships (guardrail and slopes, guardrail and ditches, etc.), the common objects were then compared to one another and adjustments were made as deemed appropriate. Severity for the sides and corners are assumed to be the same values shown for the side. "Both" means the severity for the face, corner and side impacts are the same. These objects have also been listed in the *RDG* Appendix A in order of ascending severity for each speed (40, 50, 60 and 70 mph).

There are many cases where different obstacles will appear within the clear area. Each will have its own relative severity index (e.g., a tree on a 3:1 slope, headwall and culvert opening, curb and guardrail, culvert opening and 4:1 slope). The severity table could not possibly provide a severity index for each situation. The combination of hazards adds more uncertainty as to the collision outcome. Adjustment to the severity index within the given range or even outside the range may be required.

The severity index is a very significant factor in determining user cost. Designers will need to use their best judgement in selecting a value. The sensitivity of different values should be analyzed for their impact on resulting costs. A sensitivity analysis over a range of values would be appropriate because of the variable's significance. In any case, the analyst should always apply the test of reasonableness to the output of ROADSIDE and be wary of using the results to compromise established safety practices or to justify costly or controversial new safety design practices or policies.

Actual accident history can be used to determine a cost per accident. One method for determining an average cost per accident is described in FHWA Technical Advisory T 7570.1, dated June 30, 1988. By using the SI - accident costs relationship, accident costs could be used to find a SI. As mentioned above in using actual data several gross assumptions need to be made, one of which is the model's prediction of collisions versus reported accidents. Not all collisions will result in an accident. Vehicles may drive away from an impact to a slope or guardrail. An adjustment based on a ratio of actual accidents to predicted collisions needs to be made on the SI. Additional information in this area is included in Appendix F in TRB Special Report 214.

49-10.03(10) Project Life and Discount Rate

Line	Input Data	Units
10	*Project Life*	*years*
	Discount Rate	*percent*

The project life of a roadside design is the useful life of the design and is an input value selected by the user. The discount rate is also a basic input to the economic analysis. Once these variables are selected, the program calculates the economic factors needed to complete the analysis. In the absence of other guidance, a discount rate of 4.0% is suggested.

The project life is the time period from construction to replacement of each alternative. This is also called the alternative's useful life and may have a significant effect on the analysis. There are many situations at a given location where alternatives will have different useful lives. For consistency it would be desirable to establish a common or national figure for useful lives for each alternative. Such values could not be applied at each situation because of the many uncertainties involved. It is recommended that the useful life be established for the analysis by using the best information available to an agency. Typically, 20 years is used; beyond 20 years the accuracy of the predictions is difficult to estimate. A sensitivity analysis can be used to compare different periods of time for a given location.

The discount rate usually is not a significant factor in the analysis. High rates favor future investments and low rates favor current investments. The discount rate is used to reduce various costs or benefits to their present worth or uniform annual costs so that the economics of different alternatives can be compared. If the discount rate is set equal to the real interest rate (interest minus inflation), reasonable values are in the order of 3 to 5 percent.

49-10.03(11) Highway Agency Costs

Line	Input Data	Units
11	*Installation Cost*	*dollars*
12	*Repair Cost (per accident)*	*dollars*
13	*Routine Maintenance Cost (per year)*	*dollars*
14	*Salvage Value*	*dollars*

The installation (construction), repair, maintenance and salvage value costs are the final basic inputs to the program. Once this information is provided, total present worth and annualized costs and highway agency present worth and annualized costs are computed. This is the output of the program, which enables the design engineer to make direct comparisons between several proposed alternative safety treatments.

Direct costs include construction, maintenance, repair and salvage. The most important of these costs is construction cost. Because this is a significant factor, the construction cost used in the analysis should be current and can be obtained from the latest *INDOT Catalog of Unit Price Averages for Roads - Bridges - Traffic*. A sensitivity analysis comparing variations in cost may be desirable.

Routine repair costs for a number of different types of barriers, end treatments and crash cushions are shown in Figure 49-10Q. These should be used to estimate the repair costs for these items unless better information is available.

Due to subjectivity and difficulty of determining routine maintenance costs and salvage values, the user can typically assume these to be $0 (or zero).

49-10.04 Analysis Methods

The three common methods used to compare alternative proposals in an economic analysis are as follows:

1. comparison of present worth of costs;
2. comparison of equivalent uniform annual cost; and
3. benefit/cost ratio.

When properly applied and when the results are properly interpreted, each method will lead to the selection of the same project as being the most economically advantageous. Each alternative must be compared with each of the others to determine the best selection when more than two alternatives are being compared.

In the present worth method (PW), the objective is to compare the present worth of all cash flows for a selected time period. The alternative having the minimum present worth is normally the best selection. The present worth represents the sum which would be required in the base year to finance all future expenditures (agency and user's) during the project life. ROADSIDE automatically computes the total present worth for each alternative. The analysis period for which the present worth costs are calculated must be equal for all alternatives.

In the equivalent uniform annual cost method (EUAC), all alternatives are compared on the basis of their equivalent uniform annual cost. The alternative having the minimum total EUAC is most often the selection of choice. ROADSIDE automatically computes the EUAC for each alternative. Comparison of alternatives with different analysis periods can be made. This is assuming construction replacement costs are the same in the future.

The benefit/cost ratio method measures the ratio of expected benefits to cost. These costs are usually expressed as an EUAC. The B/C ratio method is an incremental solution; i.e., it compares the differences of a pair of alternatives. Usually alternatives which include a safety improvement are compared with existing conditions (i.e., do nothing). Benefits are the reduction in accident costs (accident costs for do nothing minus accident costs for the improvement). Costs for the B/C ratio would be agency costs for that improvement.

49-10.05 Sensitivity Analysis

There are many factors which influence traffic safety policies and the development of safety programs. Rational decision-making processes combined with a cost-effective analysis are of crucial importance in the choice between competing social and economic goals. The cost-effective selection procedures provide a basic tool to compare alternative roadside improvements at site-specific locations. It was intended for evaluating improvements to either reduce the chances of a crash (remove or relocate) or reduce the severity (retrofit or shield). The decision between doing nothing and safety improvements is another question. Existing policies and standards are the overriding force in this area. ROADSIDE provides a basic tool for comparing alternative improvement options at specific locations. However, it is a probability model and the ranking of options should be viewed as a relative ranking only. Furthermore, the program is extremely sensitive to the selection of a severity index and to the costs assigned to each general type of accident.

Sensitivity is the relative effect that a variable may have on the decision. The sensitivity of each input variable on the user and agency costs are summarized in Figure 49-10R. Use of the computer program makes it relatively easy to vary an input variable. It may be desirable to test the effects of variations of the significant input variables on the selection of an alternative.

49-10.06 Examples

These examples are from the Federal Highway Administration's August 1991 *SUPPLEMENTAL INFORMATION FOR USE WITH THE ROADSIDE COMPUTER PROGRAM.* The options considered in these examples may not always correspond to those required by INDOT policy.

* * * * * * * * * *

Example 49-10.1 Culvert and protruding headwall.

Use the example problem provided in the AASHTO *RDG*, Appendix A and check the effects of changing accident costs and severity.

Design options: Option 1 - do nothing
Option 2 - shield the culvert
Option 3 - extend the culvert

Option 4 - modify culvert inlet/outlet

Sensitivity Analysis:

1. See how a change in accident costs affects the outcome (*RDG* default values vs. FHWA T 7570.1 values)

FHWA T 7570.1: Fatal accident = $ 1,500,000
Injury = $39,000 - $12,000 - $6,000
PDO = $2,000

2. See how changes in severity indices affect the outcome (*RDG* SI values vs. suggested SI values in this Section).

Summary:

1. Accident Cost. Annualized cost using *RDG* accident cost default values.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$2,060	$0	$2,060	N/A
Option 2	$858	$392	$1,250	3.1
Option 3	$225	$625	$850	2.9
Option 4	$591	$441	$1,032	3.3

Annualized Cost for FHWA T 7570.1 accident cost values.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$4,966	$0	$4,966	N/A
Option 2	$1,661	$392	$2,053	8.4
Option 3	$542	$625	$1,167	7.1
Option 4	$1,240	$441	$1,681	8.4

Discussion:

The sensitivity analysis shows that increasing the accident cost would increase the benefit-cost (B/C) ratio 2 to 3 times. The benefit (reduced accidents from existing condition - Option 1) increases for each option because of the higher relative accident cost. In most cases, using a higher accident cost will not change the order of which option has the highest

B/C ratio, but the B/C ratio may change significantly for an object with a high severity index. The example problem shows Option 4 has the highest B/C ratio when using default accident values but, when the accident costs are increased, both Option 4 and Option 2 have the same B/C ratio. The two options in either case are close enough that there is no clear cut answer. In fact, if another analysis method is used, equivalent uniform annualized cost (EUAC), Option 3 is the best choice. The user should be aware that a change in any of the input variables may alter the order of which option has the best B/C ratio. In making a decision, the analyst should obtain more information about existing practices and constraints of each option. Selection of the best option should be based on results of the model, additional information and good engineering judgement.

2. Severity Indices. *RDG* SI values in example/modified SI values in this Section (using *RDG* default accident cost).

Impact Location	Option 1	Option 2	Option 3	Option 4
Upstream side	5.5/5.4	3.0/3.4	5.5/5.4	4.0/3.2
Downstream side	5.5/5.4	3.0/3.4	5.5/5.4	4.0/3.2
Upstream corner	6.0/5.5	3.0/3.4	6.0/5.4	4.0/3.2
Downstream corner	6.0/5.5	3.0/3.4	6.0/5.4	4.0/3.2
Face	4.8/4.2	2.7/3.2	4.8/4.2	4.0/3.2

SI Selection:

Option 1 - Side: high-range of culvert >3 feet
Corner: mid-range projecting headwall >10 inches
Face: high-range of vertical wall

Option 2 - Side and corner: low-range of BCT
Face: low-range W-beam guardrail

Option 3 - Side and corner: high-range of culvert >3 feet
Face: high-range of vertical wall

Option 4 - Side, corner and face: slightly higher than high range for a 4:1 slope (10-ft embankment)

Annualized cost using different severity indices (RDG accident cost values).

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$1,629	$0	$1,629	N/A

Option 2	$1,395	$392	$1,787	0.6
Option 3	$167	$625	$792	2.3
Option 4	$310	$441	$751	3.0

Discussion:

In changing from the *RDG* SI values to the modified SI values, the following changes occur — Option 2 (shield) drops from a B/C ratio of 3.1 to be less cost-effective than the do-nothing option, Option 3 (extend) drops from a B/C ratio of 2.9 to 2.3; Option 4 (modify opening) drops from a B/C ratio of 3.3 to 3.0. Option 4 has the lowest EUAC of $751. Option 2 (barrier) has a larger exposure area than the existing conditions and, therefore, the calculated number of accidents will increase. Although the severity of the barrier is less than the existing culvert opening, the severity reduction is not enough to make the installing barrier cost-effective. If FHWA accident costs are used, the B/C ratio for Option 1 (barrier) is 2.6, Option 3 (extend) is 5.6, and Option 4 (modify opening) is 7.3.

Option 4 (modified opening) appears to be the best alternative. Constraints for this option include high potential for debris accumulation impeding water flow, soil erosion around the opening, and clear recovery area at the bottom of the slope. In selecting Option 3 (extend to clear zone), safety hazards should not be built into or around the new location (depressions, pockets, raised headwalls, humps, etc). Although Option 2 (shield with barrier) does not appear cost effective, barrier should be installed as a minimum if existing policies or practices dictate.

Example 49-10.2 Bridge Pier in Median.

Given: AADT = 30,000 with a 50% directional distribution
Growth = 4%
Design speed = 70 mph
4-lane divided highway/tangent section

Design options: Option 1 - no protection
Option 2 - W-beam guardrail with bullnose
Option 3 - concrete safety shape with tapered end section
Option 4 - concrete safety shape with sand barrels

Assumptions:

Use FHWA T 7570.1 accident cost
Project life = 20 years - 10 years for gravel barrels (Option 4)

Discount rate = 4%

No salvage value, except concrete safety shape (Option 4) where salvage value is approximately equal to new installation cost

Sensitivity Analysis:

1. See how changes to accommodate lane distribution affect the outcome.

   a. without lane distribution

   b. with lane distribution - run program separately for each lane (Figure 49-10E);
   c. use 30%-70% lane distribution; 4,500 (median lane) - 10,500 (right lane);

   d. with lane distribution - run program with user factor adjustment;

   e. use 0.62 (between 0.64 and 0.60 in Figure 49-10F).

Calculations:

Input Variable	Option 1	Option 2	Option 3	Option 4
Lateral distance (A)	35′	29′	34′	28′
Long. length (L)	50′	130′	210′	100′
Width (W)	3′	15′	5′	15′
Installation cost	$0	$10,000	$7,000	$17,000
Repair cost	$0	$100/acc	$0	$1000/acc
Maintenance cost	$0	$20/year	$10/year	$100/year
Salvage value	$0	$0	$0	$5,000
Severity index (face)	6.5	4.0	3.8	3.8
Severity index (side)	6.5	4.6	4.8	3.3

Summary:

Annualized cost without accommodating for lane distribution.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$24,486	$0	$24,486	N/A

Option 2	$12,154	$1,528	$13,682	8.1
Option 3	$10,938	$1,050	$11,988	12.9
Option 4	$5,154	$3,614	$8,768	5.4

Annualized cost with lane distribution - program run separately for each lane.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$15,946	$0	$15,946	N/A
Option 2	$8,012	$1,528	$9,540	5.2
Option 3	$7,152	$1,050	$8,202	8.4
Option 4	$3,426	$3,576	$7,002	3.5

Annualized cost with lane distribution - adjusting with user factor.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$15,180	$0	$15,180	N/A
Option 2	$7,536	$1,522	$9,058	5.0
Option 3	$6,780	$1,050	$7,830	8.0
Option 4	$3,174	$3,594	$6,768	3.3

Discussion:

Accident, agency and total equivalent uniform annual cost (EUAC) are shown for each option. The B/C ratios compared with no protection (Option 1) are also shown. The computer printout shows agency and accident cost for one direction. These costs are doubled assuming the other side of the piers are treated the same for both directions and the piers are in the center of the median.

Changing the analysis method to accommodate lane distribution lowers the B/C ratio for each option. The accident and agency costs are higher without lane distribution, because the model assigns 15,000 ADT to the lane closest to the obstacle (in this case the median lane). In adjusting for lane distribution, the EUAC are lower because most of the traffic will be in the right lane. This is an additional 12 feet further and therefore less probable of reaching the obstacle. EUAC and B/C ratios are slightly different between the user factor method and running the program separately for each lane. The analyst could easily check the sensitivity between methods by changing the user factor. The range would vary between running the model without lane distribution (user factor = 1.0) and with the lane distribution (user factor = value in Figures 49-10F and 49-10G).

All three improvements are cost effective compared with the no-protection alternative. Option 3 (concrete safety shape with tapered end section) has the highest B/C ratio. Option 4 (concrete safety shape with sand barrels) has the lowest EUAC. Each of these options may have other advantages and disadvantages which should be investigated before making the final decision.

Example 49-10.3 Ditch Along Roadside of 4-Lane Divided Highway

Determine the most cost-effective alternative.

Given:
AADT = 13,000 with a 50% directional distribution
Growth = 2%
Design speed = 70 mph
4-lane divided highway/tangent section

Design options:
Option 1- no protection
Option 2- W-beam guardrail
Option 3- install pipe and regrade to 6:1/6:1 ditch section

Assumptions:
Use FHWA T 7570.1 accident costs
Project life = 20 years
Discount rate = 4%
No salvage value
User factor 0.89 to accommodate lane distribution

Sensitivity Analysis:

1. Maintenance has pipe in stock and can do Option 3 with a 20% savings. See how a change in installation cost affects the outcome (Option 3a).

2. See how a change in accident cost affects the outcome (*RDG* default values - FHWA T 7570.1).

Input Variable	Option 1	Option 2	Option 3
Lateral Distance (A)	35′	29′	34′

Long. Length (L)	50′	130′	210′
Width (W)	3′	15′	5′
Installation Cost	$0	$10,000	$7,000
Repair Cost	$0	$100/acc	$0
Maintenance Cost	$0	$20/year	$10/year
Severity Index (Face)	6.5	4.3	4.3
Severity Index (Side)	6.5	4.8	4.8

Summary: Annualized cost - FHWA T 7570.1 accident costs.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$3,913	$0	$3,913	N/A
Option 2	$2,507	$759	$3,266	1.9
Option 3	$2,410	$525	$2,935	2.9
Option 4	$2,410	$422	$2,832	3.6

Annualized cost - *RDG* accident costs.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$1,581	$0	$1,581	N/A
Option 2	$1,117	$759	$1,875	0.6
Option 3	$1,081	$525	$1,606	1.0
Option 4	$1,081	$422	$1,503	1.2

Discussion:

In changing from the FHWA T 7570.1 accident costs to the *RDG* accident costs, the following occurs: The decrease in the accident cost decreases the benefit-cost ratio by a factor of 3. The benefit (reduced accidents from existing condition - Option 1) decreases for each option because of the lower relative accident cost. In most cases, using a lower accident cost will not change the order of which option has the highest B/C ratio, but the B/C ratio may change significantly for an object with a high severity index. In this case, Option 3a has the highest B/C ratio with either set of accident costs.

If the equivalent uniform annualized cost (EUAC) method is used, Option 3a is still the best choice. In fact, using the *RDG* accident costs, Options 2 and 3 are both less desirable than Option 1. Only Option 3a has an EUAC less than Option 1.

As mentioned in the previous examples, each option may have other advantages and disadvantages that should be studied before making the final decision. Selection of the best option should be based on the results of the model, additional information and good engineering judgment.

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49-10.07 Application of ROADSIDE to Non-Level Roadsides (Slope Correction for Cost-Effectiveness Calculations)

Figure 49-2A provides the recommended clear zone ranges for various design speeds and for various side slope conditions. It also recommends different ranges for various traffic volumes, but this is a cost-effectiveness consideration rather than a safety need.

Using the information, a series of factors have been developed to input into the ROADSIDE computer program to better describe the effective lateral clearance (the "A" dimension).

It would then seem logical that, to achieve the same degree of safety and probability of accidents, the relationship between required clear zone distances could be used to develop factors to be multiplied to the actual lateral offset distance to derive the effective lateral clearance.

Assuming that the ROADSIDE program assumes a relatively flat side slope, the "flatter than 6:1" columns would have a correction factor of 1.0. The values in the other columns would become the denominators, and the values in the "flatter than 6:1" columns would become the numerators. The resulting fraction would be the factor to multiply the actual lateral clearance by to get the effective clearance.

Using the methodology described above, the factors become as follows:

Clear Zone Adjustment Factors		
Design Speed	Cut Slopes	Fill Slopes

	3:1	4:1	5:1	6:1	Flatter Than 6:1		6:1	5:1	4:1
≤40	1.0	1.0	1.0	1.0	1.0	1.0	0.93	0.93	0.81
45-50	1.23	1.14	1.0	1.0	1.0	1.0	0.89	0.80	0.62
55	1.33	1.25	1.11	1.0	1.0	1.0	0.91	0.83	0.67
60	1.63	1.44	1.18	1.08	1.0	1.0	0.87	0.81	0.65
65-70	1.56	1.27	1.17	1.08	1.0	1.0	0.88	0.82	0.67

When the ROADSIDE program asks for the lateral distance, A, one would multiply the plan or actual distance by the slope correction factor to get the effective lateral clearance. For example, a fixed object located 16 feet off the traveled way on a 5:1 fill slope on a highway with a design speed of 45 mph would be effectively 16′ x 0.80 or 12.8′ away. The 12.8′ should be the value used for cost-effectiveness calculations.

49-11.0 ASSUMPTIONS FOR EMBANKMENT WARRANT FIGURES

Figures 49-4B through 49-4G present warrants for guardrail on embankments based on embankment heights, slopes and design year ADT’s. These figures were developed using the computer program ROADSIDE, as described in Section 49-10.0. Section 49-11.0 discusses the variables and assumptions that were used to develop Figures 49-4B through 49-4G. The line numbers listed below refer to the line numbers for imputing data into ROADSIDE; see Figure 49-10B. Because the program uses English units, a soft conversion of the metric units was used (e.g., 100 km/h = 62.2 mph, 3.6 m = 11.81′). The following steps were used in the calculations:

1. <u>Guardrail Calculations</u>. ROADSIDE was first used to determine the present worth of providing guardrail along a 300-m embankment. In addition to the following, Figures 49-11A through Figure 49-11F provide several of the assumptions used to develop these figures:

    a. Line 2. Figures 49-11A through 49-11F present the design year traffic volumes selected by the Department. The current traffic volumes were used in the program. A 2% traffic growth factor per year was assumed.

    b. Line 3. The calculations were run assuming a 2-lane, undivided facility with 3.6-m wide travel lanes.

    c. Line 4. The roadway was assumed to be on a tangent and in level terrain.

    d. Line 6. The English equivalent of the metric design speed was used.

e. Line 7. The lateral location of the guardrail from the edge of the travel lane was assumed to be 3.0 m for ADT's between 700 and 1500 and 3.6 m for ADT's greater than 1500. The longitudinal length of the guardrail was calculated to be 300 m + 2*L_R, where L_R is from Figure 49-5F. The width of guardrail was assumed to be 0.6 m.

f. Line 9. The severity indices from Figures 49-10H and 49-10 I for the guardrail face and the terminal ends were interpolated for the metric design speeds. The metric interpolations are shown in Figure 49-11G, Metric Severity Indices. It should be noted that for ADT's less than 6000 and design speeds of 80 km/h or less, a buried end terminal was used. For ADT's 6000 or greater and design speeds greater than 80 km/h, a FHWA approved proprietary guardrail end treatment (CAT) was assumed. No corner impacts were assumed.

g. Line 10. The project life for the guardrail installation was assumed to be 10 years with a 4% discount rate.

h. Line 11. The installation cost varies according to the design speed and ADT; see Figures 49-11A through 49-11F. Installation costs were taken from the *INDOT Catalog of Unit Price Averages for Roads - Bridges - Traffic*.

i. Line 12. The repairs costs in Figure 49-10Q were used.

j. Line 13. No maintenance costs were assumed.

k. Line 14. No salvage value was assumed.

2. <u>Embankment Calculations</u>. ROADSIDE was also used to determine an equivalent embankment severity index for an embankment without guardrail. The severity index for the embankment was selected to match the present worth of the guardrail using the assumptions in Figures 49-11A through 49-11F and the following:

a. Line 2. Figures 49-11A through 49-11F present the design year traffic volumes selected by the Department. The current traffic volumes were used in the program. A 2% traffic growth factor per year was assumed.

b. Line 3. The calculations were run assuming a 2-lane, undivided facility with 3.6-m wide travel lanes.

c. Line 4. The roadway was assumed to be on a tangent and in level terrain.

d. Line 6. The English equivalent of the metric design speed was used.

e. Line 7. The lateral location of the embankment from the edge of the travel lane was assumed to be 3.0 m for ADT's between 700 and 1500 and 3.6 m for ADT's greater than 1500. The embankment was assumed to be 300-m long. For calculation purposes, the width of the embankment was assumed to be 7.5 m.

f. Line 9. For embankments, the severity index was selected to match the present worth for the guardrail installation.

g. Line 10. The project life for the embankment was assumed to be 20 years with a 4% discount rate.

h. Line 11. No installation costs were assumed because the embankment would also be in place for guardrail installations.

i. Line 12. No repairs costs were assumed.

j. Line 13. No maintenance costs were assumed.

k. Line 14. No salvage value was assumed.

3. Slope Equivalents. Using Figure 49-10K and interpolating for the metric design speeds, the slope indices for various slopes were developed and are presented in Figure 49-11G, Metric Severity Indices. The higher indices were assumed to be for embankment heights of 5.0 m or higher. The mid-range indices were assumed to be for heights of 2.0 m. The lower range indices were assumed to be for embankment heights of 0.5 m. Using Figure 49-11G and the equivalent embankment severity indices shown in Figures 49-11A through 49-11F, the equivalent slope could be determined for each embankment height and ADT.

4. Data Plotting. The data points determined in Step 3 were used to develop Figures 49-4B, through 49-4G. Figure 5.1, on page 5-3 of the AASHTO *Roadside Design Guide*, was also imposed on the charts as a lower boundary for when guardrail would be required. In addition, the 18,000 ADT was assumed to be the maximum traffic volume that could be reasonably obtained on a 2-lane facility and, therefore, is considered to be a lower boundary.